THE UNIVERSITY OF ALBERTA

GRAVEL RIVERS WITH LOW SEDIMENT CHARGE

by

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ABSTRACT

An attempt is made to find the rules that correlate the channel geometry of gravel rivers and gravel canals with discharge and bed material. Five gravel rivers, which flow out of lakes, were investigated a short distance downstream of the lake outlets. Four laboratory tests were made to extend the ranges of discharge and grain size. Use was also made of gravel river and gravel canal data available in various publications. The relation between tractive force and sediment size, a fourth-root flow formula and a modified version of Lacey's width-discharge relation, were used to derive a set of regime equations for gravel rivers and gravel canals with low sediment discharge and low depth. Methods of sampling coarse river beds are discussed and it is concluded that areal grid samples, worked out by number in terms of the equivalent spherical diameter, or in terms of the intermediate axis, can be used as approximations to the standard sieve curve of the surface layer of a gravel paved channel.

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Rolf Kellerhals.

GRAVEL RIVERS WITH LOW SEDIMENT DISCHARGE

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (cont'd.)

LIST OF FIGURES (cont'd.)

LIST OF PICTURES

LIST OF PICTURES (cont'd.)

CHAPTER 1, INTRODUCTION

1.1 THE PROBLEM

Almost all the large rivers in Alberta and in British Columbia flow on gravel or boulder beds during the major part of their course in these two provinces. Interferences such as dams, diversions and dikes, are becoming more and more frequent as the development of natural resources progresses towards and into the mountainous areas at the headwaters of these rivers. To appreciate the consequences of interferences, the factors governing the equilibrium or regime of gravel rivers ought to be known. It is the purpose of this thesis to explore one special aspect of this problem, the equilibrium for low sediment transport rates.

A number of gravel rivers below natural lakes were studied during the summer of 1962. During the following winter a few flume experiments were made to investigate the no-transport equilibrium of gravel channels. Based on the results of this work an attempt will be made to describe the characteristic properties of a gravel river with low sediment discharge. It is hoped that this study will help in predicting the changes that take place in a gravel river downstream of a newly constructed dam.

1.2 NOTATIONS AND DEFINITIONS

1.2.1 Notation. All letter symbols used in this thesis will be defined where they first appear. For convenience, the more commonly used symbols and abbreviations will be listed in Appendix A.

1.2.2 Sand, Gravel, Boulders. Material consisting mainly of grains coarser than 2 mm will be defined as gravel. Sand will mean material predominantly in the grain size range from 0.06 mm to 2 mm. It is usually obvious whether the bed material of a particular river reach is either sand or gravel. River beds consisting mainly of the intermediate material in the 1.5 to 4 mm range are uncommon, (see Chapter 5).

No upper grain size limit will be imposed on gravel. The term "boulder" will be reserved for single, exceptionally large stones.

1.2.3 The Sediment Load. The part of the total load that moves by sliding, rolling or saltation, will be called bed load. The remaining part is carried in suspension and will be called suspended load. The fraction of the bed load that consists of material similar to that of the river bed will be referred to as bed material load, and the remaining fraction as wash load.

1.3 THE CONCEPT OF REGIME

1.3.1 Definition. What is meant here by the synonymous expressions of "river in equilibrium", or "river in regime", probably needs some clarification, since there has been considerable argument about these concepts, (Ref. 1, 2, 3, 5).

Regime only applies to alluvial river reaches. Such a reach is considered to be in regime or in equilibrium, when its average dimensions such as slope, width, depth, meander wave length, etc., do not change appreciably over periods of many years. The reach should be a few meander lengths long. The average dimensions may oscillate about a certain value, but they should not exhibit a definite trend. If meander cutoffs occur

in the reach under consideration the river slope may oscillate about a mean value with periods of 50 or more years.

1.3.2 <u>The Ideal Regime River</u>. The idealized, perfect regime river has to be visualized as flowing across a broad, very long and gently sloping plane; built up by the river itself in earlier times. The downstream end of this plane is formed by an undestructible barrier, across which the river discharges at supercritical velocity. The state of equilibrium of any reach along this river will depend almost entirely on the functions of time that describe the water and the sediment load (water sediment complex) entering the reach at this section; the possible exceptions to this rule are factors such as climate and vegetation along the reach.

Somewhat simplified this can be expressed as:

$$S_{Pl}, S_R, \text{Sec.}, W_m, L_m = \text{fns.}\ (Q, Wa, C, Sed) \qquad 1.3.2.1$$

Where

S_{Pl}	Slope of the plane	average values along a reach.
S_R	Slope of the river	
Sec.	Cross section of the river	
W_m	Width of the meander belt	
L_m	Meander wave length	
Q	Discharge	Functions of time.
Wa	Physical properties of water such as density, viscosity, temperature.	
C	Sediment discharge	
Sed	Physical properties of sediment such as size distribution, resistance to abrasion, etc.	

The width of the river or the width of the meander belt are in certain cases independent variables; examples being rivers meandering in restricted channels that were cut under different geological conditions, flume experiments, and rivers trained between rigid banks.

Equation 1.3.2.1 is simply a list of possible causes and their effects. It does not imply that all the independent variables shown actually influence all or any of the dependent variables. Experience with unlined irrigation canals has proven conclusively that a relation similar to, but simpler, than equation 1.3.2.1 (no meandering, independent variables essentially constant with time), is unique within narrow limits, (Ref. 2,5,6,7,8,9). Sediment discharge was not measured in most of the canals mentioned in the above references. The bed load was usually very small but the suspended load of some canals was considerable.

Geologists, until recently, evaded the question of dependency by talking of a graded river and defining it as a river in which the slope is adjusted to transport, with prevailing channel characteristics, the sediment supplied at the head of the graded reach. (Ref. 1.)

1.3.3 Application of the Regime Concept to Rivers. Steadily changing geological and climatic conditions make the perfect state of equilibrium, as described in 1.3.2, a rather unlikely occurrence. Particularly at present, only a few thousand years after an ice age, ideal equilibrium cannot be assumed lightly for the majority of alluvial river reaches.

The concept of the regime river has become valuable to

engineers and geologists in spite of these limitations for the following reason: a river that aggrades or degrades usually does so very slowly, or in other words, the trends mentioned in 1.3.1 will be slow. On a long time average the sediment load increase or decrease, along a reach of a few meander lengths, is small compared with the load transported through that reach. This probably accounts for the near impossibility of distinguishing between aggrading, degrading and regime rivers on the basis of channel geometry and hydraulic measurements not covering periods of several life times. (Ref. 4, p.45-49.) Summarizing, one may say that most rivers are not truly in equilibrium but their channel geometry and hydraulic characteristics can be expected to resemble the equilibrium state closely.

Work done by the U.S.G.S. (Ref. 10), suggests that equation 1.3.2.1 is applicable to sand rivers, as far as Q as an independent variable is concerned. For the majority of rivers investigated the sediment discharge was not available.

For gravel rivers the uniqueness of the relation stated in equation 1.3.2.1 has never been established, particularly where width as a dependent variable is concerned. The equilibrium may depend mainly on the way in which it was reached. The geological history of such rivers is therefore important.

1.3.4 Regime of Rivers below Lakes. A river without sediment load will degrade its bed eventually, even if it consists of coarse gravel that does not move at the highest floods. Chemical and physical agents will disintegrate the stones. In addition, if climatic factors cause the discharge of such a river to decrease, it might not be able to change its channel to suit

the new conditions. This means that river reaches just below large lakes downstream to the first sediment supplying tributary, may not be in regime, even if they are alluvial.

CHAPTER 2 THE FIELD WORK

Some general aspects of the field work will be described in paragraphs 1 to 8. In paragraphs 9 to 15 the relevant data of each measuring reach will be presented separately. The important measurements of all the reaches are summarized in Tables 2 to 9.

2.1 THE SELECTION OF SUITABLE RIVERS

For the proposed study of the low-transport equilibrium, a gravel river had to meet a rather large number of limiting conditions, which are stated below.

a) In order to make the assumption of a low sediment transport equilibrium possible, suitable study reaches had to be on rivers that flow out of lakes, preferably not more than a few miles downstream of such a lake.

b) As pointed out in 1.3.1, the study reaches had to be alluvial. To have a small tributary stream join the main river just upstream of the study reach was desirable, particularly if the tributary had a small discharge and a large sediment load. (See 1.3.4)

c) The discharge, at the time when measurements were made, had to be known, and a comparison between this flow and the general flow characteristics of the river had to be possible. This meant that the rivers had to be gauged and that flow records of at least twenty years had to be available.

d) The ideal river would have had a fairly constant discharge all year round, but this could obviously not be expected. Based on the assumption that gravel rivers with very little sus-

pended sediment and small bed load charge are morphologically inactive at low stages, it was specified only that the river should have a small ratio between floods of high, and floods of low frequency. The question of what discharge should be considered responsible for a given river channel (dominant discharge), loses some of its importance, if this type of river is studied.

e) The rivers had to be reasonably free from human interferences to ensure equilibrium conditions.

f) Reasons of time and money imposed some limitations on the distances that could be travelled to obtain field data.

g) Only river reaches accessible at least by a four-wheel drive vehicle could be considered. Financial reasons, and the weight of the equipment excluded access by plane or boat.

With the help of maps, and after consulting the Alberta Water Resources Branch, it was concluded that no river in Alberta meets the requirements outlined above sufficiently to justify studying it.

The author knew vaguely of some possibilities in British Columbia. These were confirmed through correspondence with the B.C. Water Rights Branch, and with the aid of aerial photographs. A considerable number of rivers in Central B.C. south of Quesnel appeared to be possible. Finally, the Chilko, Taseko, Quesnel and Cariboo Rivers, below the respective lakes of the same names, were chosen. Fig. 1 shows their general location. It was left to be decided in the field, exactly what reaches would be studied. Not all the conditions specified above were satisfied by each river, but they all met the important conditions, 2.1.b and d, alluvial reach and similar flood discharges each year,

very well indeed. All four rivers are within one hundred miles of Williams Lake, so that travel between them did not consume an unreasonable part of the time spent in the field. Storage dams have been considered by various agencies on all the lakes mentioned above. Detailed maps of the lake outlet areas were therefore available.

Measurements made by the B.C. Water Rights Branch on the Thompson River below Kamloops Lake will also be included in this study. The author spent only one day there in late September, sampling the bed material.

2.2 ORGANIZATION OF FIELD WORK

Each river was visited twice; first in June or July, and again in late August or September. The main objectives of the first visit were the selection of one or two suitable measuring reaches, and the measurement of water surface slope, and water velocity at high stage. During the second visit to a river in fall the bed material was sampled and cross sections were measured at most stations.

The field work would have been simplified considerably if the rivers could have been visited first at low stage.

2.3 SELECTION OF STUDY REACHES

Along measuring reaches the rivers had to be reasonably straight and had to have a well defined channel. Dense bush and difficulties of access usually limited the number of possible measuring reaches. For the same reasons most of the reaches that were finally selected were not as long as would have been desirable for obtaining a representative slope. This is certainly one of the most severe limitations of this study.

Fortunately three of the six slope measurements could be tied into the much longer slope surveys made by the International Pacific Salmon Fisheries Commission and by the B.C. Hydro and Power Authority. The close agreement is reassuring.

2.4 WATER SURFACE SLOPE AND SURFACE VELOCITY

Once a reach was selected, three to six stations were established along it at fairly equal intervals. These stations were marked on both sides of the river with blazes, or with flagging, and numbered consecutively in a downstream direction. The distance across the river between corresponding station markers was measured by triangulation or by stadia. The distance along the river bank was obtained by chaining. On one side of the river, bench marks were established at each station and tied into each other. Water surface elevations were usually measured twice at each station, once during the first visit to a river and once during the second visit in fall. The water surface velocity was obtained at about two locations along each reach. It was measured by timing a float along a short distance.

2.5 SAMPLING THE BED AND BANK MATERIAL

All the river beds had a very distinct pavement, consisting of the coarsest stones available from the bed material. Diameters ranging from 3 to 15 inches were most commonly encountered. This pavement material was sampled by picking stones at random, and measuring their three axes. This areal grid sampling technique was first described by Wolman in Ref. 11, and was used extensively by Qureshi (Ref. 12). Wolman only measured the intermediate axis of each stone. This is justified if all the samples are of reasonably equal roundness. However the

samples that were collected for this study had widely varying average shapes. To take this into account all three axes of each stone were measured, as described by Qureshi in Ref. 12. Random picking appears to be the only possible method of sampling very coarse material, if the dredging and sieving of samples as large as a few cubic yards is excluded for financial reasons.

All the random picking of stones from under water was done by walking over the area to be sampled, making long steps with closed eyes, and after each step picking up the stone that happened to be under the toes. In this way the sample area was covered by a zigzag course. Usually the general direction was kept constant. A sample of 60 to 80 stones covers then an area of about 3 by 100 feet. Sometimes a broader and shorter area was sampled by moving back and forth across it, taking care not to cover any particular place more than once. Between 50 and 80 stones per sample would have been desirable. Usually the effect of the very cold water resulted in smaller samples.

Even at low stage in fall, most of the rivers were so swift that it was impossible to collect stones from a depth greater than two feet. Fortunately all the rivers, except the Taseko, had very clear water. The bed material could be seen all across the channel, either from shore or from a boat, and an estimate could be made of how well a particular sample represented the bed pavement of the central part of a channel.

Most rivers were quite low during the second visit in September. Often places could be found above water, where the pavement appeared to be very similar to that existing in the main channel. This was always a welcome sight as the discomfort

of picking stones from two feet of cold, swift, water could then be avoided. Random sampling above water was done by stretching a survey tape across the sampling area and picking up stones below every foot-marker. A few samples were collected by picking and measuring all the stones larger than some minimum value, which were exposed within a certain small area.

The volume and equivalent spherical diameter (diameter of sphere of equal weight), were calculated, using the formulas given in Ref. 12.

$$V = c^2 (a \ b)^{\frac{1}{2}} \qquad 2.5.1$$

$$\phi = \left(\frac{6}{\pi} c^2 \ a^{\frac{1}{2}} \ b^{\frac{1}{2}}\right)^{\frac{1}{3}} \qquad 2.5.2$$

Where a, b, and c, are the largest, intermediate, and smallest axis of the stone; V is the volume and ϕ is the diameter of a sphere of equal weight. Fig. 2 shows a useful nomogram for calculating V and ϕ . The rather unusual selection of units results from the fact that a, b, and c, were measured with a piece of survey tape, but it was more convenient to have ϕ in inches, to facilitate comparison with other studies.

Equation 2.5.1 could only be verified in a few cases, where the stones of a sample were small enough to be weighed on a small balance. The results are shown in Table 1 together with some data from Ref. 14. The weight was calculated for a specific gravity of 2.65. Agreement appears reasonably good. Considering that ϕ only depends on the third root of V, one can expect ϕ to be accurate to within 10 percent. It is probably a few percent too small, but the scanty data did not justify the use of a correction coefficient in equation 2.3.1.

Frequency curves (Log ϕ vs. "percent by number finer than") were plotted for each sample. These curves are included in this

report (see 2.9 to 2.15). The more important gravel samples were also plotted as "logarithm of intermediate axis vs. percent by number finer than", as suggested by Wolman. Only one of these curves is shown here (Fig. 49), but the results of analysing the samples in this fashion can be seen in Table 2.

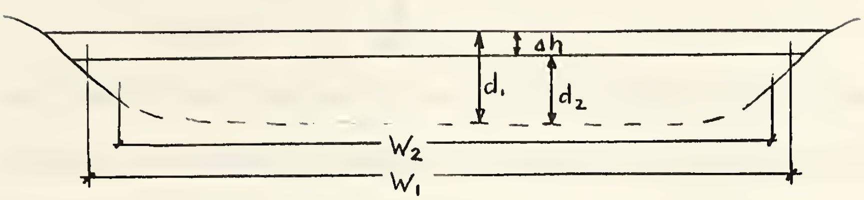
The sampling technique described above is comparatively new and not generally accepted. It will therefore be discussed briefly in Chapter 4.

At a few favourable places, such as road cuts, samples of the bank material were taken with a shovel. This material was split and sieved wet, down to mesh number four. The finer fraction was taken to Edmonton, and sieved to mesh 200. The methods used for sampling and wet sieving were rather crude, so that the resulting sieve curves may not be representative.

2.6 CROSS SECTIONS

At almost all stations cross section measurements were made with level and rod, and by wading out into the river as far as possible. The cross sections usually extend from a point well above bankfull stage to a similar point across the river. The sections are complete only at those stations where the boat could be used for echo sounding, or where a bridge or cable spans the river. The missing parts of the cross sections were estimated, using field book entries on the apparent depth and shape of the main river channels, measured surface velocities, and, where feasible, the Manning equation. This equation, applied to the water surface slope measurements at high and low stage, gives, under suitable circumstances, a good estimate of the missing water depth data.

The sketch below shows a typical river cross section. The dashed line indicates the missing parts.



Index 1 refers to high stage, index 2 to low stage.

Manning's equation gives:

$$Q_1 = \frac{1.49}{n} W_1 d_1^{\frac{5}{3}} S^{\frac{1}{2}} \qquad \text{(high stage)}$$

$$Q_2 = \frac{1.49}{n} W_2 (d_1 - \Delta h)^{\frac{5}{3}} S^{\frac{1}{2}} \qquad \text{(low stage)}$$

Divided by each other and rearranged:

$$\frac{d_1}{d_1 - \Delta h} = \left(\frac{Q_1}{Q_2} \cdot \frac{W_2}{W_1} \cdot \sqrt{\frac{S_2}{S_1}} \right)^{\frac{3}{5}} \qquad 2.6.1$$

All the terms in equation 2.6.1, expect d_1, are known. As no width to depth ratios of less than 15 were encountered, hydraulic radius and wetted perimeter were usually replaced by an average width, and by the depth of the central part of the channel, so that the product width times depth is equal to the section area. Depth estimates with equation 2.6.1 were made for known sections to test this method. Very close agreement was reached, when Δh was larger than about $\frac{d_1}{5}$ and when the section was trapezoidal, and in a straight reach. (See 2.13.) It should be kept in mind that some of the estimated sections are based on scanty data, particularly the Quesnel and Taseko sections.

In future d will stand for the average channel depth in

the central part of the river, as shown above. The gravel rivers studied were found to have fairly flat beds over a major part of the width, so that d is usually well defined.

d* will stand for

$$d^* = \frac{A}{W_s} \qquad 2.6.2$$

where W_s is the water surface width and A the cross section area. In shallow wide rivers, d* is only a few percent larger than the hydraulic radius.

2.7 DISCHARGE RECORDS

With the exception of the Taseko, all the rivers that were studied had gauging stations very near the measuring reach. Flood frequency curves (annual maximum daily mean discharge vs. frequency of occurrence) were plotted on logarithmic probability paper. (Fig. 3). The same data were also plotted on arithmetic probability paper. The logarithmic plots are given here, because their fitting lines are considerably straighter.

2.8 REGIME CONSIDERATIONS

In order to facilitate the comparison of the equilibrium state in different rivers, a certain typical discharge has to be assigned to each river. What discharge can be considered as typical poses a difficult question. Possibly there is no completely correct answer, as the varying discharges of a river might produce a channel that could not have been obtained by a certain constant flow. In Ref. 10 the problem was solved by using discharges of equal frequency of occurrence on all the rivers, without trying to decide whether these discharges actually could have formed the channel. In this study more is known about the recent geological history of the measuring

reaches than in Ref. 10, therefore an attempt has been made to select representative or dominant discharges that could have formed the particular channel.

Some of the study reaches are very close to lake outlets. Almost no sediment is supplied to the river upstream of these reaches, so that the bed and bank erosion and the flattening of the slope caused by extreme floods cannot be repaired. The typical or dominant discharge is therefore a flood of low frequency. As pointed out in 1.3.4, there is no guarantee that equilibrium exists at all, even during an extremely high flood.

Other measuring reaches are located several miles downstream of lakes. Tributaries supply sediment to the river between the lake outlet and the study reaches. Here the erosion caused by an extreme flood can to some extent be repaired by the river. An extreme flood lasting perhaps a few weeks would also not be able to flatten the river slope. At dominant discharge such reaches must just be able to transport the coarsest fractions of sediment entering from tributaries. Floods with a recurrence interval of from 2 to 5 years were considered to be representative of these reaches. Whenever feasible, the highest discharge at which measurements had been made was taken as the dominant discharge.

Instead of using a dominant discharge estimated on the basis of frequency considerations, some people prefer to work in terms of a "bankfull discharge", determined from cross sections and gauge-discharge relations. A number of good reasons can be put forward for doing so, particularly if the river is meandering in a valley with distinct flood plains. The author does not believe that the concept of "bankfull discharge" is

useful when dealing with incised stable channels shortly below lakes (e.g. Thompson River below Kamloops Lake, Cariboo River below Cariboo Lake). Presumably bankfull discharge for these channels would have to be taken as the discharge at which the water reaches the apparent high water line, as shown by soil, vegetation and driftwood. One wonders whether the elevation of this line is not primarily a function of the climate, the type of vegetation along the river and the discharge regime of the river. However, the author's estimate of dominant discharge is almost identical with bankfull discharge for reaches that lie several miles below lakes such as "Chilko River at Henry's Crossing", or "Quesnel River at Lawless Creek", etc. Bankfull discharge will be given separately, wherever it differs appreciably from the assumed dominant discharge.

An estimate of the typical cross section of each measuring reach at dominant discharge was also made. These estimates are based mainly on the cross section measurements. On occasions, an inspection of the river both upstream and downstream of the measuring reach has also been taken into consideration.

2.9 THE QUESNEL RIVER AT LAWLESS CREEK

Description of the River Upstream of the Measuring Reach. The Quesnel River originates in Quesnel Lake (area: 105 sq. m.) on the western slope of the Cariboo Mountains. The Dominion Water Resources gauging station, 8KH1, (Quesnel River at Likely, B.C.), is situated just below the lake. At low stages the remnants of a small dam can be seen in the river near the gauging station. This dam was built in 1898 and used for drying up the river in winter so that gold could be mined from its channel. The high stages of the Quesnel River were never greatly influ-

enced by the dam. About half a mile below the gauging station the river enters the Red Canyon. The reach between the remains of the dam and Red Canyon is not suitable for measurements as the river bed there consists of angular boulders up to several feet in diameter.

The Red Canyon is about 5 miles long. Huge boulders or possibly bedrock are exposed in the river. The tailings of a placer mine, (Bullion Mine), were once discharged into the river at a point approximately half way down Red Canyon. The Bullion Mine was started in the 19th century. Until 1922 it operated intermittently, but from 1922 to 1942 it was in full operation, discharging around 800,000 cu. yards of sand and gravel into the river each year. This amounts to a sediment charge of about 5-20 parts per million during the summer months. Since 1942 the mine has been closed down.

Flow Records. The general flow characteristics of the Quesnel River can be seen from Fig. 3 (flood frequency), Fig. 6 (daily discharges of a few years), and Fig. 7 (flow duration). All these plots are based on the 38 years of records for the gauging station 8KH1 at Likely, B. C.

The Measuring Reach. A measuring reach was selected at the downstream end of Red Canyon. There the river flows through a circular, flat area which is almost completely surrounded by steep cliffs, (Fig. 4 and 5). Pictures 1 and 2 were taken from a position high on these cliffs.

The river has recently been degrading along the measuring reach. Comparison between Picture 1 and the air photos taken in 1955 (Fig. 5), illustrates this. The air photos were taken at a discharge of 10,000 cfs. and show practically the whole

gravel flats flooded; yet on picture 1 a discharge of 14,800 cfs produces no spill. The river has gone down about 4 feet in 7 years. This degradation appears to be caused by the suspension of operations at the Bullion Mine in 1942. The behaviour of the Dominion Water Resources gauge 8KH3 on the Cariboo River near Quesnel Forks supports this conclusion, (See 2.10.).

Cross Sections and Slope. The cross sectional data are very scanty, (Fig. 8), due to two unforeseen circumstances. The river could not be crossed by boat, even in fall, and there was no other feasible access to the left side of the measuring reach. The water surface width was measured by triangulation, so that the estimated position of the left banks and of the bar in section 4, should be accurate to within a few feet. The sections appeared trapezoidal. Depth at each section was estimated in the field and these estimates were checked with equation 2.6.1. Fig. 9 shows the water surface slopes as measured in July and in September. Approximate energy lines are also plotted. A summary of the channel geometry is given in Table 3.

Bed and Bank Material. The results of the bed and bank material sampling are listed in Table 2a and are plotted on Fig. 10. Picture 3 shows the area of sample 4. The bed material in the central part of the channel is fairly well represented by sample 1. Samples 4 and 5 were taken in an old channel that no longer carries water, even at high river stage. Below station 4 the bed material appears to become somewhat finer. All the samples contain predominantly very well rounded material.

Regime Considerations. It may seem paradoxical in a study of the equilibrium of gravel rivers, to use measurements taken on a river that is known to be degrading. In the author's

opinion the following points justify the assumption that this measuring reach closely resembles a state of equilibrium:

(a) The river has formed its channel by digging into its own deposits.

(b) An estimate of the volumes involved will show that at high stage the bed load of actual bed material, (bed material load), must be smaller than 1 ppm. At low stage no bed material transport takes place.

(c) The laboratory work,(Chapter 3), showed how rapidly a state resembling equilibrium becomes established in the upper part of a flume after the feeding of sediment is suddenly stopped.

(d) The argument put forward under 1.3.3 applies to this reach.

The dominant discharge will be taken as 14,800 cfs. Slope and section measurements have been made at this discharge. No bed load transport could be heard while doing it. This discharge corresponds to a 3 year flood.

The estimate of an average section is given on Table 3. It is based on sections 1 to 4, because some distance below the measuring reach the river flows again in a channel with a similar cross section. It is assumed that the wide channel at sections 5 and 6 is caused by the much finer bank material, (sample 3). Bank erosion in this part of the reach is shown on Fig. 8, section 6. This is the only erosion that could be detected along the measuring reach.

2.10 THE CARIBOO RIVER NEAR QUESNEL FORKS

Description of the River. Until recently the Cariboo River was called Quesnel North Fork. It originates in Isaac Lake (13 sq. m.), on the western slope of the Cariboo Mountains. After a course of approximately 60 miles it joins the Quesnel River at Quesnel Forks. On the way it flows through Lanezi Lake (6 sq. m.), Sandy Lake (4 sq. m.), and Cariboo Lake (4 sq. m.). From approximately 2 miles below Cariboo Lake, to a point 1 mile upstream of Quesnel Forks, the river is deeply incised.

Measurements were made along two reaches. The upper one at the outlet of Cariboo Lake will be described in 2.11.

The Measuring Reach. Fig. 4 and 5 show the lower study reach near Quesnel Forks. It was selected because the river there meets the requirements of 2.1 reasonably well. The major limitations of this reach are:

- It is very short.
- It is not straight.
- Since it is 20 miles below Cariboo Lake, sediment transport at high stage is considerably greater than in the other reaches.
- At low stage the water surface slope is not uniform. Near station 3 the main flow shifts from the right side to the left side. The channel is wider and shallower there and rapids form at low stage.

The pictures 4 and 5 were taken along this reach.

Flow Records. The Dominion Water Resources gauging station 8KH3 (Cariboo River near Hydraulic) is right on the measuring reach. The gauge is located at station 5, and the measuring section is identical with cross section 1. Fig. 3 (flood frequency)

and Fig. 13 (flow duration), have been prepared from the 36 years of records available for this station. Fig. 11 and Fig. 12 show some daily mean discharges.

Sections and Slope. The cross sections are plotted on Fig. 14. At station 1 the cross section below water is based on recent soundings by the Dominion Water Resources Branch. At station 4 the cross section has been obtained by sounding off an old cable car. The slope measurements are shown on Fig. 15. The river slope at high stage is probably best represented by the energy line st. 1 to st. 4 of June 29, 1962. A summary of the Hydraulic Measurements is given in Table 4.

Bed and Bank Material. The gravel samples are summarized in Table 2b and are plotted in Fig. 16. The pictures 6 and 7 give some idea of the appearance of the river bed and banks. Samples 2 and 5 represent the bed pavement. It appeared that the majority of the stones that were measured for these two samples were moved very recently. There was no growth of algae on them. Only a few stones were as well rounded as the material that had been sampled along the Quesnel River. Samples 1, 3 and 4b represent the pavement along a line 3 to 4 feet below bankfull stage. Sample 4a was taken about 1 foot below bankfull stage. Fig. 17 shows the sieve curves of two samples taken along this reach.

Regime Considerations. An average cross section of the Cariboo River in this area is shown ob Table 4. It is derived from:

(a) Cross section measurements.

(b) An inspection of the river above the measuring reach.

(c) Aerial photographs.

The dominant discharge will be taken as 12,000 cfs based on the following reasons:

(a) Slope and sections have been measured at 12,000 cfs.

(b) The two year flood is only 700 cfs larger.

(c) Some bed material was probably in motion at 12,000 cfs. The sound of rolling stones could be heard at intervals of a few seconds.

Specific Gauge. Specific Gauge curves were plotted for the Water Resources gauge 8KH3 at the downstream end of the measuring reach. From 1926, when the gauge was installed, until 1935, no change took place. From 1935 to 1949 specific gauges went up steadily. The 1949 stage-discharge curve is about 1 foot above the original one over a range of discharge from 5000 cfs to 12,000 cfs. After 1949 specific gauges went down again and appear to have stabilized at about 0.2 feet below the 1949 stage-discharge curve.

2.11 THE CARIBOO RIVER AT THE OUTLET OF CARIBOO LAKE

A general description of the Cariboo River is given in 2.10.

The Measuring Reach. Fig. 18 and Fig. 19 show the upper study reach on the Cariboo River. Rollie Creek has pushed the river to the left side of the valley. At present Rollie Creek flows into Cariboo Lake. The area marked Rollie Creek Delta on Fig. 19 is densely wooded. No recent channel of the creek could be found there, but the air photos and old gold diggings show that the area belongs to its delta. Pictures 8 and 9 were taken along this reach.

Flow Records. The discharge measuring section of the Dominion Water Resources gauging station 8KH13 lies within the

study reach. The gauge is situated on Cariboo Lake near Keithley Creek. It was installed in 1961. Fig. 12 shows the 1962 record of this station together with the record of the gauging station 8KH3 near Quesnel Forks. From a comparison of all the peak flows that occurred during the summer of 1962, it appears that those measured at the outlet of Cariboo Lake are about 12% smaller than the corresponding ones at Quesnel Forks. With this correction factor the flood frequency curve of Fig. 3 can be used for this reach.

The highest discharges of Rollie Creek are estimated to be in the order of 500 cfs.

Sections and Slope. The cross sections are shown on Fig. 20. Section 2 is the Water Resources discharge measuring section. The actual river only begins approximately 200 feet further downstream. Between section 2 and section 3 the river is shallow for a short distance. At low stage some white water forms. No bedrock could be discovered in the river, but the deepest parts of the channel could never be seen. Energy line and water surface slope are given on Fig. 21. The flow velocity at station 1 is less than 0.5 ft s . The water elevations at this station can, for all practical purposes, be considered lake elevations. The water surface slope, st. 2 to st. 7 of July 10, is probably most representative of the river slope at high stage. A summary of the section and slope measurements is given in Table 5.

Bed and Bank Material. The gravel samples are summarized in Table 2c and are plotted on Fig. 23. The pictures 10 and 11 will give some idea of the shape and arrangement of the bed

pavement. Sample 5 represents the central parts of the bed; samples 3 and 4 the gently sloping left side of the channel; and samples 1 and 2 the right side, about 3 feet below bank-full stage. The stones of all the samples were covered with algae and seemed to have been at rest for many years. At first glance the stones on the right side appeared well rounded, but on closer inspection approximately half were found to be split. This is probably a result of freeze-thaw action, (picture 11). On the left side rounded, flat and angular stones were mixed. The author has no explanation for the presence of the angular stones.

Fig. 22 shows three sieve curves of material taken from the Rollie Creek delta.

Some Regime Considerations. The argument put forward in paragraph 1.3.4 applies to this reach. It lies immediately below a lake and no sediment load is supplied to it at the lake outlet. The river is therefore unable to repair any erosion caused by extreme floods. A state of equilibrium could only exist during such a high flood. The dominant discharge is taken as 17,000 cfs, which corresponds approximately to the fifty year flood. The maximum discharge on record is 20,000 cfs in Quesnel Forks, which corresponds to around 17,000 cfs at this measuring reach. The dimensions of an average section at dominant discharge are given in Table 5. Depth has been obtained by extrapolating the stage discharge curve of the Water Resources gauging station.

2.12 THE TASEKO RIVER BELOW TASEKO LAKE

Description of the River. The Taseko River has its origin on the eastern slope of the Coast Range of British Columbia.

After flowing in a NW direction for about 10 miles it enters Taseko Lake (total area 13 sq. m.). The delta of the Tchaikazan River splits Taseko Lake into two parts, the upper and the lower lake.

After leaving the lower Taseko Lake, the river takes a northerly course across the semi-arid Chilcotin Plateau. Fifty-five river miles downstream of the lake outlet the Taseko River joins the Chilko River. In summer the discharge of the Taseko River is almost all glacial melt water, carrying a very fine, whitish silt load. The Taseko Lakes are not deep enough for this silt to settle out. Therefore, even downstream of the lake, one cannot recognize any features of the river bed through more than approximately 1 foot of water. Taseko, incidentally, means "white water" in the language of the Chilcotin Indians.

The Measuring Reach. A short reach was studied on the Taseko River one mile downstream of the outlet of Taseko Lake. Fig. 24 is a sketch of the area and on Fig. 25 the lake outlet can be seen in stereo. The situation is similar to that at the outlet of Cariboo Lake. The delta of Beece Creek has pushed the Taseko River to the left side of the valley. On this delta Beece Creek splits up into many channels. Some discharge into the river, others into the lake. In the summer of 1962 almost all the channels carried water, and some new ones were in the process of being formed. The larger channels have a bed of coarse gravel with some boulders.

At the upper end of the measuring reach the river is somewhat obstructed by large boulders that either fell off the mountainside to the left, or were brought down by avalanches. An old cable car crossed the river in the centre of the measuring reach, but the installation collapsed when an attempt was

made to use it for an echo sounder traverse.

The measuring reach is not typical of the Taseko River in this area, due to the presence of a long bar on the left side. However dense bush made it necessary to take advantage of this rather unusual bar for levelling and chaining.

Pictures 12-14 show this reach.

Flow Records. Only intermittent flow records of the Taseko River are available. They are for the summers of 1929 and 1930, and were measured at Nemaia Crossing, 15 miles downstream of the lake outlet. There are also about 30 discharge measurements available. They were made between 1929 and 1949, both at Nemaia Crossing and at the lake outlet. An estimate of the Taseko discharge can be made in the following manner. The Chilko River is gauged at the outlet of Chilko Lake (8MA2), and forty miles further downstream just below the confluence with the Taseko River (8MA1). After about mid-June, the summer-dry Chilcotin Plateau contributes only slightly to the discharge of the Taseko and the Chilko River. For the summer months, one can therefore estimate the discharge of the Taseko River at Taseko Lake from the difference between the two Chilko records. Comparison with the existing Taseko records indicates that by taking 93% of the difference of the two Chilko records, a good estimate is obtained of the Taseko River discharge at the lake outlet. No estimate of the Taseko River discharge can be made for May or for the first half of June. This is no great handicap, since the existing Taseko records and information obtained locally, indicate that Taseko Lake usually reaches its highest stage only in July or August. Knowing that the lake is 4,300 feet above sea level and that the drainage basin is heavily

glaciated, this is not surprising.

The flood frequency curve of Fig. 3 and the daily discharges plotted on Fig. 26, are based on the two Chilko records. The correction factor mentioned above has not been applied to them.

The discharge of Beece Creek has been measured occasionally. A maximum of 520 cfs is on record.

Section and Slope. Cross sections are plotted on Fig. 27. Depth has been estimated with equations 2.6.1 and from field notes. It was possible by wading out from the right bank to obtain depth measurements in the main current. It is unlikely that the sections are much deeper further out. Air photos and inspection of the river indicate that only section 2 can be considered as typical of the Taseko River just below the lake. Its dimensions for August 21, 1962 are listed in Table 6.

The slope measurements are shown on Fig. 28, together with the river slope as it appears on a large scale map of the B.C. Hydro and Power Authority. The rapids near the large boulders (picture 14), appear as the steep drop at the upstream end of the slope plot. Station 0c is well above these rapids. The water surface at station 3 is high because the water piles up against the upstream end of the bar. The water surface slopes st. 1 to st. 5 of August 13 and 21 are probably representative of the river's slope.

Bed and Bank Material. A summary of the bed material samples is given in Table 2d and the same samples are plotted on Fig. 29. The pictures 12 - 14 show some of the areas that were sampled. Samples 2, 5 and 6, are believed to represent the bed material of the deeper parts of the channel. Sample 3 represents the shallow left side of the channel and

sample 1, the steep banks of the right side. The majority of the stones were well rounded.

Two samples were collected with a shovel on the right bank. The sieve curves are shown on Fig. 30. These samples should give some idea of the material that Beece Creek brings into the Taseko River.

Regime Considerations. This reach lies only a short distance below a lake. The sediment load, particularly the bed material load, must be extremely small. Since bed erosion caused by extreme floods can only be repaired slowly, the assumption of a fairly high dominant discharge seems justified. It will be taken as 6,400 cfs, which corresponds to the once in ten years flood. A discharge of 5,400 cfs has been measured at Nemaia Crossing.

The dimensions of an average section are given in Table 6.

2.13 THE CHILKO RIVER AT HENRY'S CROSSING

Description of the River. The Chilko River originates in Chilko Lake (area 70 sq. m.) on the eastern slope of the Coast mountains of British Columbia. From Chilko Lake the river flows in a north westerly direction across the Chilcotin plateau. After a course of approximately 50 miles, it joins the Chilcotin River near Redstone, B.C. Fig. 31 is a sketch of the first nine miles of the Chilko River, showing the two reaches that have been studied. (See also 2.14).

Two miles downstream of the narrows that mark the outlet of Chilko Lake, the river is very wide and deep. This reach of approximately 1 mile can, for all practical purposes, be considered as a small lake. Between the two lakes the Chilko River is

wide and slow flowing. There are a few islands in the channel, but one could hardly call the river braided. This section of the river appears to be stable. No traces of recent river erosion could be seen. A measuring reach was selected between the two lakes. It will be described in 2.14 (Fig. 41).

After leaving the lower lake, the Chilko River enters a valley which resembles the river valleys of central Alberta. It is between 600 and 2000 feet wide, and 100 to 200 feet deep. Air photos show terraces and abandoned river channels at various elevations above the present river. (Fig. 32). A comparison of the curvature of the valley sides with the meander pattern of the river, suggests that a larger river than the present one has done the major part of the valley-cutting work. Along the first ten river miles only a few places of active river erosion could be detected on the aerial photographs. Even in places where the river impinges on a steep valley side there appears to be little erosive activity. Air photos show trees and bushes growing along the river on the outside of such curves.

Approximately 5 miles below Chilko Lake, Lingfield Creek joins the Chilko River. This creek is almost dry during the summer months, but in spring it might have a discharge of 200 to 300 cfs. It is steep and carries a heavy load of coarse gravel (picture 15).

From the outlet of the small lake (3 miles below Chilko Lake) to the mouth of Lingfield Creek, the Chilko River has one distinct channel. Downstream of Lingfield Creek the river is partly braided. The ratio of talweg length to valley length varies from 1.2 to 1.4, with a maximum of 1.6 along one mile.

Chilko means "clear water" in the language of the Chilcotin

Indians. (Chilcotin - people of the clear water.)

The Measuring Reach. The lower study reach on the Chilko River begins about 8 miles below Chilko Lake and ends 3,600 ft. further downstream near the bridge of Henry's Crossing (Fig. 31 and Fig. 32). The reach is fairly straight. Along most of the length the Chilko has only one channel, but there is a distinct tendency towards braiding. The pictures 16 and 17 were taken along this reach.

Flow Records. The Dominion Water Resources gauging station, 8MA2, (Chilko River near Tatla Lake, B.C.), is located at the outlet of Chilko Lake. Thirty-four years of records exist for this station. Fig. 3 shows the flood frequency curve of this station. On Fig. 33 the mean daily discharges of the last three summers have been plotted. The summer of 1962 was unusually wet over the whole Chilcotin plateau. Therefore the July discharges of the above gauging station have been increased by 5% for use on this measuring reach.

Sections and Slope. The cross sections are plotted on Fig. 34 and 35. Unfortunately most of them are incomplete. Depth was estimated with equation 2.6.1. The general shape of the bed could be seen clearly from the banks. The section at station 5 was obtained on August 18, 1959, by engineers of the International Pacific Salmon Fisheries Commission (IPSFC). They spanned the river with a cable and towed an echo sounder across. This section was used for testing equation 2.6.1. The depth calculated from this equation is 0.25 ft. greater than the depth obtained by dividing the area of the section by an average width, (128 ft.). At the bridge the river has been obstructed by material that was pushed into it from the left.

The water surface slope measurements are plotted on Fig. 36. On Fig. 37 the river slope (water surface) between Chilko Lake and Henry's Crossing is shown. It has been measured for the B.C. Hydro and Power Authority for a power development study. Water surface elevations were obtained at all the points for which the talweg length is shown on Fig. 37. For comparison the slopes that were measured by the author and by the IPSFC are also shown.

Table 7 gives a summary of the channel geometry.

Bed and Bank Material. A summary of the bed material samples is given in Table 2e and they are plotted on Fig. 38 and 39. On Fig. 38 all the samples that were collected by using some grid system (walking or with tape), are plotted against frequency of exceedence by number. On Fig. 39 the samples that were collected over a certain area are plotted by weight and by number. Samples 2, 3, and 5, are most representative of the river bed. They are unfortunately all small samples, and are therefore unreliable.

Fig. 40 shows the sieve curves of two gravel samples that were taken from the banks of this reach.

The pictures 16 to 18, illustrate the location and the appearance of several gravel samples.

Regime Considerations. The discharge of this river was unfortunately quite low when it was visited first in July 1962. At 3,600 cfs no coarse gravel was being transported. Had transport been taking place it would undoubtedly have been heard. Based on the arguments put forward in 2.7, the dominant discharge will be taken as 5,600 cfs which corresponds to a 5 year flood. The dimensions of an average section at this discharge

are listed in Table 7. Depth has been calculated from Manning's equation.

2.14 THE CHILKO RIVER AT THE OUTLET OF CHILKO LAKE

A general description of the Chilko River and of the flow records available for it is given in 2.13.

The measuring reach. The second study reach on the Chilko River lies between Chilko Lake and the small lower lake, (see Fig. 31 and Fig. 41). This reach was selected because the bed pavement appeared to be finer than in any other study reach, and because the International Pacific Salmon Fisheries Commission (IPSFC) had surveyed the river along this reach. (The Chilko Sockeyes spawn in this area). At the upstream end of the measuring reach there are a few large boulders in the channel. At low stage some white water forms there.

Pictures 19 shows this reach.

Sections and Slope. The cross sections are shown on Fig. 42. The main channel was sectioned by echo sounding. The author can offer no explanation for the wide shallow bench along the right bank of the river.

The slope measurements could be tied into the slope survey of the IPSFC. All the water surface slope measurements are shown on Fig. 43. The IPSFC slope was measured with stadia at all those places where a steep river bank made levelling tedious. This should explain the strange "hump" near station 2, as the left bank is steep there.

The slope and section measurements are summarized in Table 8.

Bed Material. The results of the bed material sampling

are summarized in Table 2f and plotted on Fig. 44. Samples 1 and 2 were taken near the outside edge of the shallow bench on the right side of the river. Viewed from a boat, the bed material of the main channel appeared to be somewhat coarser. A ϕ_{50} of 4 inches will be assumed for that part of the channel. Practically no angular stones could be found, but in each sample about one quarter of the stones were split, probably as a result of frost action.

Regime Considerations. This reach lies only a short distance below a lake. Therefore a dominant discharge of low frequency of occurrence seems justified. It will be taken as 7,000 cfs, which corresponds to a 50 year flood, and is 200 cfs greater than the greatest flood on record. Using Manning's equation, the dimensions of section 1 at dominant discharge have been calculated. The results are listed in Table 8.

2.15 THE THOMPSON RIVER BELOW KAMLOOPS LAKE

Engineers of the B.C. Water Rights Branch have recently been investigating the possibilities of lowering the flood levels in Kamloops City by improving the flow conditions at the outlet of Kamloops Lake. A large-scale map of the first 4 miles of the Thompson was prepared. Fig. 45 is a copy of the key plan accompanying this map. Seventeen cross sections were obtained by echo sounding. The water surface slope was unfortunately only measured at low stage and with stadia.

The Measuring Reach. Along the first mile below the lake the river sections show extreme variation both in area and depth and the water surface slope is not uniform. Therefore only the reach from sections 7 to 17 (10,000 ft along ℄ of river), has been used in this study. The channel is reasonably

straight and deeply incised (see Fig. 46). The steepness of the banks suggest that the river is degrading (see picture 20). Bedrock is exposed in a few spots on the left side of the channel. The river bed seems to consist of coarse gravel. The possibility of bedrock exposures on the river bed cannot be excluded.

Flow Records. The Thompson River is gauged near Spences Bridge, some 60 miles below Kamloops Lake. (Thompson River near Spences Bridge, 8LF51). Forty-seven years of records exist for this station. In 1960 another gauging station was installed about 1 mile below Kamloops Lake. (Thompson River near Savona, 8LF33.) Comparison between the records of the two stations indicates that the discharges near Savona are approximately 5% smaller than near Spences Bridge.

The flood frequency curve of the Thompson River near Spences Bridge is shown on Fig. 3. Based on this curve the fifty year flood at the outlet of Kamloops Lake has been estimated as 135,000 cfs and this will be taken as the dominant discharge. The flow duration curve of the Thompson River near Spences Bridge is shown on Fig. 47.

Sections and Slope. The water surface slope along the measuring reach is as uniform as one could expect from stadia measurements made at low river stage. The overall slope st. 7 to st. 17 should be reasonably representative of the river slope at high stage. It is 0.72 parts per thousand.

On Fig. 48, 4 typical sections (9, 10, 15 and 16) have been plotted. Section 15 is similar to sections 8, 12 and 14.

Section 9 is similar to sections 10 and 11.

Section 16 is similar to section 13.

Section 10 is similar to sections 7, 11 and 17.

From the rating curve of the gauging station 8LF33, it has been estimated that the water surface at dominant discharge is 17 feet higher than the water surfaces shown on Fig. 42. On all the sections the vegetation line is indicated. The water reaches this line at a discharge of approximately 110,000 cfs, which corresponds to the five year flood. A summary of the cross sections at dominant discharge is shown in Table 9, together with some other relevant information about this measuring reach.

Bed and Bank Material. The bed pavement was sampled on a bar near section 10. Equivalent spherical diameter and intermediate axes are plotted on Fig. 49. The sample is also listed in Table 2. The author is unable to say to what extent the gravel pavement on this particular bar resembles that in the deeper parts of the river. The B.C. Water Rights Branch party, who surveyed the river, believes that the channel is paved with this type of material, but how coarse it is, they could naturally not say. Picture 20 shows the bar on which the gravel pavement was sampled and picture 21 is a close-up.

Two samples were taken with the shovel; one of gravel from the river bank near station 4,l.b. and one of sand from a protected little beach near station 5, r.b. The sieve curves are shown on Fig. 44.

CHAPTER 3 LABORATORY WORK

3.1 OBJECT

During the winter 1962-63, several flume experiments were made, to study the no-transport equilibrium of gravel channels. The main objects of these tests were:

(a) to see whether the no-transport equilibrium of gravel is unique.

(b) to observe the formation of a gravel-paved channel.

(c) to observe sand and gravel transport over both sand and gravel beds.

(d) to see whether a small laboratory river could be induced to dig a channel in gravel similar to the channels observed in the field.

3.2 EXPERIMENTAL SETUP

As natural a gravel as possible was to be used for the experiments, but the equipment available in the hydraulics laboratory imposed some limiting conditions on the type of material that could be employed. From the experimental data of Meyer-Peter (unpublished) it was concluded that in order to obtain a no-transport equilibrium at sub critical flow, and at a discharge of 1 to 2 cfs (explained below) the gravel had to be seived through a ½-inch mesh. The laboratory has no special dirty water system, therefore, to keep the main water system reasonably clean, the fines also had to be taken out of the gravel.

Samples of all the major gravel pits in the vicinity of

Edmonton were sieved out. Three typical sieve curves are shown on Fig. 51. In all of them the grain sizes between 0.7 mm and 5 mm were found to be deficient. This was not desirable for the laboratory work but it had to be accepted. Ten cubic yards of the gravel shown in Fig. 52 were purchased. In the laboratory this material was then sieved through a $\frac{1}{2}$-inch mesh and washed on a 50 mesh. Sieve curves of the final product are shown in Fig. 53. As can be seen the washing operation did not meet with success.

A glass walled flume, 60 feet long, 4 feet wide, and 3 feet deep, was used for the experiments. It was set at a slope of 0.5 percent for all the experiments. In the first test equilibrium was to be approached by allowing the flow to build up a gravel bed in the flume. For this run a short inlet chute was installed at the upstream end of the flume. The gravel was then fed into this chute by means of a conveyer belt. The flume was obstructed at intervals of a few feet with a piece of plywood, 2 inches high, to start the building up process on the smooth flume bed. The flow also had to be kept away from the smooth walls in order to simulate natural conditions. This was done by placing pieces of plywood, 4 inches wide, along the sides. Some coarse gravel was also put into the flume in little piles along the edges before the first run. It could then be spread out along the glass walls where necessary.

Discharge was measured with an orifice meter in the overhead pipe between the pump and the stilling pond at the head of the flume.

As the general aim of all the experiments was to reproduce conditions similar to those encountered in the field, discharge

was limited to one or possibly two cfs by the width of the flume.

The picture 22 shows the experimental setup before the first experiment.

3.3 THE EXPERIMENTS, PROCEDURE AND RESULTS

3.3.1 Test 1. The object of the first experiment was to approach a no-transport equilibrium state by building up a gravel bed. This is conparable to the situation on the river reach "Quesnel at Lawless Creek", where the river bed has been built up by the Bullion Mine tailings.

A flow of 1 cfs was started in the flume and gravel was fed into it at a fairly steady rate of one shovel full every five seconds. This amounts to approximately 0.3 pound per second, or to a charge of 0.5 percent. Picture 23 shows the installation during this building-up process. Picture 24 was taken after 2 hours. It shows how the building up proceeded down the flume in a delta like manner, with a steep front, 2-3 inches high, (visible in the centre of the picture). Some sand that was carried in suspension past this delta front was deposited further down the flume in ripple formation. Flow over the gravel bed was supercritical, but below the delta front it was sub critical.

The building up process was continued for 5 hours, until the delta front came to within a few feet of the tail-gate. By then the upper two thirds of the reach were mainly gravel, interspersed with sandy patches (picture 25). The sand moved along this reach as sheet flow, practically all transport taking place close to the bed. Across the predominantly gravel areas the

gravel rolled along the bottom, coming to rest occasionally. On sandy patches the gravel grains appeared to move faster and more easily and rarely came to rest. The deposits in the upper parts of the flume were firm. They had a gravel skeleton with sand filling the interspaces.

In the lower third of the flume the bed was predominantly sandy (picture 26). This reach was somewhat influenced by backwater from the tailgate. The majority of the gravel grains that entered this reach rolled across it and were deposited at the steep delta front. The deposits along this reach were soft and obviously did not have a gravel skeleton.

Just before stopping gravel feeding, the water surface slope was measured at five foot intervals, in the centre of the flume and at the quarter points. Then the flume was immediately drained; pictures of the bed were taken (25 and 26) and the bed elevations were obtained in all the places where the water surface elevations had been measured. Picture 26 shows the point-gauge used for measuring water surface and bed elevations.

A flow of 1 cfs was then resumed, but no further gravel was supplied. After two hours of degrading transport was almost at a halt, and the bed was distinctly paved with the coarsest fractions of the gravel. The flume was then drained and bed elevations were obtained as before. At the start of degrading the tailgate had been raised 0.13 ft. to provide more room for the deposition of material at the lower end of the flume. This shortened the significant reach (unaffected by entrance and exit conditions) from 35 ft. to 25 ft.

To ensure that a state of equilibrium had been observed, a flow of 1 cfs was again started and left running for twenty-four

hours. At that time absolutely no transport could be seen. Water surface elevations were again measured, then the flume was drained and bed elevations were obtained. No change greater than 0.02 ft. had taken place during the last 24 hours of degrading. Picture 27 shows the pavement that had formed on the degraded bed.

The author expected that the flow would cut a distinct channel in the gravel deposits during degradation, but this did not happen. However the depth became somewhat less uniform across the channel. The variation was greatest half way down the flume where depth was 0.18 ft. on the right side and 0.10 on the left side. Average water surface and bed elevations before and after degrading are shown on Fig. 54.

3.3.2 Test 2. As the first test had not produced an equilibrium cross section resembling a natural channel, gravel was lifted into the flume and a channel was constructed on the degraded bed of Test 1 (picture 28). The channel was 4 inches deep and 15 inches wide. No banks were built in the lower third of the flume. A flow of 1 cfs was started and left running for 24 hours. After that time no transport could be seen. In the upper third of the channel a gravel river with gravel banks had formed (picture 29). Further downstream the channel had widened and the depth had decreased, until, at the end of the artificial banks, the flow filled again the whole width of the flume as in Test 1. This widening was caused by the abrupt ending of the artificially installed banks.

Fig. 55 shows the average water surface slope after equilibrium has become established, together with the bed elevations

along the centre line of the flume before and after the test. Two cross sections, measured 15 and 20 ft downstream of the flume inlet, are also shown.

3.3.3 <u>Test 3</u>. The object of the third test was to obtain a no-transport equilibrium in a different way than in Test 1, in order to establish, whether the resulting equilibrium would be identical with that obtained in the previous test.

The gravel was shoveled onto a very steep slope (Fig. 56). A discharge of 1 cfs was then started. This experiment is comparable to the situation than exists at the outlet of a lake, which is dammed by glacial debris.

Initially the bed degraded very rapidly and antidunes formed in the lower half of the flume. After 24 hours all transport had come to a standstill. Water surface and bed elevations were then measured as in Test 1. The results are plotted on Fig. 56.

3.3.4 <u>Test 4</u>. A final test was made by running a discharge of 2.25 cfs over the bed, as left from Test 3. Again no transport could be seen after 24 hours of flow, when the flume was dewatered. Water surface and bed elevations were measured as in Test 1 and are plotted on Fig. 56.

During this test the baffle piers that had been installed below the inlet chute were washed out. Some of the coarse gravel that had been used to pave the stilling basin was moved 10 to 20 ft. down the flume. Therefore the steep water surface slope at the upstream end of the flume should be disregarded.

CHAPTER 4 WOLMAN'S METHOD OF SAMPLING GRAVELS AND THE ROUGHNESS OF CHANNELS

During the course of the field work the method of gravel sampling proposed by Wolman (Ref. 11) was used extensively. (See paragraph 2.5 for a description of the procedure.) There are two different ways of establishing the sampling grid and the results can be worked out in several fashions. As the method is relatively new, no particular procedure has become entrenched. Therefore the author feels justified in putting forward his opinion about the proper use of this technique.

4.1 THE AIMS OF A GRID SAMPLING TECHNIQUE

It is accepted standard practice to characterize the grain size distribution of non-cohesive soils by taking a small volume, sieving it out in square mesh sieves and plotting the results as mesh size vs. percent of total dry weight passing through that mesh (or sometimes: vs. percent of total dry weight retained by that mesh). Since this kind of grain size distribution curve has been used for practically all the work done on rivers, the aim of Wolman's grid sampling is to provide a fast, inexpensive approximation to the sieve curve of the top layer of a gravel deposit. The top layer is usually considered to consist of all the material between the surface and the deepest exposed particles, (See Fig. 57), (Ref. 11, 12, 14, 15).

4.2 ESTABLISHING A GRID

Wolman suggested two methods of establishing a grid; either to use actual lines and to collect the stones vertically below certain points (stretched out survey tape, checkerboard arrange-

ment of wires, etc.), or to make random steps picking up the stones that happen to fall under the tip of one's foot. Qureshi only used the second method. In this study both methods have been used. Only the random step approach is feasible for sampling under water, and on dry ground it is almost impossible to obtain true random samples by stepping, particularly if the average diameter of the stones is comparable to the length of a human foot.

Unfortunately the two different grids do not produce similar results. Table 2 and Figs. 23 and 29 show clearly that the grid by stepping produces samples with a considerably smaller logarithmic standard deviation, $\left(2\sigma_{(\log\phi)} = \log\left(\frac{\phi_{84}}{\phi_{16}}\right)\right)$. There is no reason to doubt that the samples collected by both methods are reasonably unbiased. The different results are caused by the fact that the operator's foot does not enter into small gaps between stones. However, if a grid point happens to fall above a gap, the usually small stone from the bottom of the gap will be collected (Fig. 58).

The curves (log b or log ϕ vs. percent by number finer than) produced by the two different grids do not differ greatly in the coarser part of a sample. As this range of the curve is the most important one, at least for certain applications (see 4.5), it seems justified to continue using both methods. For sufficiently large samples collected with a fixed grid the following statement may be made: "If f percent (by number) of the stones in a sample are finer than ϕ_f, then f percent of the area is covered by material finer than ϕ_f." This statement is not true for samples collected by pacing.

4.3 THE INTERPRETATION OF SAMPLES

4.3.1 The Frequency. Wolman takes it for granted that the samples have to be plotted against frequency by number, yet Qureshi (Ref. 12) used frequency by weight for many of his curves. The following points will show that the use of frequency by weight is very time consuming, inaccurate, subjective, and also does not produce the desired results as stated under 4.1.

1. As a rough approximation to a natural gravel, one can assume a material consisting of only two grain sizes, D = 1 inch and D = $\frac{1}{2}$ inch, each size accounting for 50% by weight, (Fig. 59). A sample of N stones collected from this material by using a fixed grid will probably contain roughly $\frac{N}{2}$ stones of each grain size. If this sample is then analyzed by weight the median diameter and even the ϕ_{15} will be 1 inch, while a plot against frequency by number will produce a curve similar to the sieve curve of the material.

2. If grain size is plotted against frequency by weight the resulting curve depends almost entirely on the few largest stones in the sample. On river beds one often encounters the occasional boulder that does not really belong to the bed material. It might be an erratic boulder that fell into the river in the course of meander migration. If such a stone happens to be in a sample that was plotted by weight, then the resulting sieve curve might easily indicate grain sizes a few hundred percent in error. To overcome this difficulty the stones greater than a certain limit would have to be rejected, but this introduces a subjective factor into the sampling technique.

Ref. 12 suggests a rule for finding the maximum size that should be included in a sample, but this rule is so complicated that one could hardly expect a field party to apply it successfully. It is also not based on much evidence and does not exclude personal judgement completely.

If a few oversized stones happen to be in a sample that is plotted by number the effect is almost negligible. Wolman's paper shows how well samples plotted by number can be reproduced by different observers.

3. A further point against plotting samples by weight is the amount of work involved. If an easily measured quantity like the intermediate axis is plotted against frequency by number the samples can be analyzed in the field by untrained operators, particularly if a simple number of stones like 49, 99 or 199 has been collected. (The frequency f of the i-th stone in a sample of n stones is $\frac{i}{n+1}$. The use of n + 1 instead of n in the denominator facilitates plotting on probability paper.)

4.3.2 The Linear Grain Size Parameter. The size of the intermediate axis determines whether a grain is retained or passes through a given round hole sieve. The same is almost true for square mesh sieves, except that the minor axis (c) has a slight influence. Wolman (Ref.11) therefore recommends to measure only b of each stone and to use it as the linear parameter of grain size. Krumbein and Pettijohn state in Ref. 16 that the equivalent spherical diameter Φ and b, usually lead to almost identical results. Qureshi in Ref. 12 used Φ for some curves and c for others. He justifies the use of c by its predominant influence on Φ according to equation 2.5.2. To interpret the results he

obtained by plotting c, he uses the relation $\phi_f = 1.55\ c_f$. Since the proportionality constant obviously depends on the type of gravel that is sampled, and since c by itself has very little influence on the ordinary volumetric sieve curve of a gravel, the author does not consider the use of c justified. In this study all the major gravel samples have been analyzed by using ϕ and b. The results are listed in Table 2 and are discussed separately for each river in Chapter 2. The ϕ curves are reproduced because of the work involved in constructing them. The b curves look very similar. A typical one is shown on Fig. 49. Table 2 indicates that the logarithmic standard deviations are almost the same for corresponding b and ϕ curves. ϕ over b ratios at constant frequency ($\frac{\phi_{50}}{b_{50}}$, $\frac{\phi_{65}}{b_{65}}$ etc.) range from 0.8 to 1.03 and stay fairly constant for a given river reach. Considering that the ϕ's of equation 2.5.2 are somewhat small (Table 1), a good average ratio is 0.95.

Whether ϕ or b should be given preference as a grain size parameter is difficult to decide. b is more closely related to the standard sieve curve. From the point of view of a worker interested in sediment transport, ϕ offers some advantages such as its close relation to the weight of a stone and the fact that it is smaller than b for very flat stones. Such stones rarely ever have their largest face exposed to the full force of the current. Therefore ϕ might characterize the exposed surface more adequately than b.

For future work the author recommends, as Wolman did previously, working in terms of b. The great amount of labor involved in calculating ϕ is not justified by the somewhat doubtful advantages of this parameter. All three axis should be

measured on a few typical samples to have some record of the shape of the material.

4.4 COMPARISONS BETWEEN AREAL AND VOLUMETRIC SAMPLES

Wolman concludes on the basis of sieve curves (by weight) for volumetric samples and frequency (by number) plots for areal samples, that areal grid sampling generally produces similar curves but indicates larger grain sizes. Qureshi gives sieve curves and areal samples that were taken along the same river reach. The sieve curves apply to the top layer as shown in Fig. 57. The material was collected above water at low river stage. The areal samples were collected from below water by stepping. The average D_{50} based on approximately 15 sieve curves is 1.6 inches, D_{65} is 1.9 inches. The average ϕ_{50} of the eleven samples for which ϕ or c curves by number were calculated is 1.95 inches and ϕ_{65} is 2.4 inches. The agreement seems very close when the following facts are taken into consideration: The volumetric samples were taken higher up in the channel, where the material is likely to be finer, and the samples were collected by walking rather than on a fixed grid. (See 4.2). The curves that were obtained by plotting against frequency by weight give ϕ values several times larger than the corresponding D values of the sieve curves.

4.5 THE USE OF AREAL SAMPLING FOR ESTIMATING FRICTION IN ROUGH CHANNELS

The following paragraph is based mainly on Ref. 17 and Ref. 18.

Friction of rough turbulent flow is commonly described by either an exponential equation of the form:

$$v_m = c_1 R^{c_2} S^{c_3} \qquad 4.5.1$$

or by a logarithmic equation of the form:

$$\frac{1}{\sqrt{f}} = c_4 \log \frac{c_5 R}{k_s} \qquad 4.5.2$$

where R = hydraulic radius
c_i = constants
f = friction factor = $\frac{8gRS}{v_m^2}$
k_s = Nikuradses equivalent sand grain roughness.

Since it is well established that the energy losses in fully turbulent flow over rough boundaries are closely proportional to v_m^2 , equation 4.5.1 can be rewritten:

$$v_m = c_6 \left(\frac{R}{k_s}\right)^{c_7} v_* \qquad 4.5.1a$$

v_* is an abbreviation for $\sqrt{gRS}$ and is usually called the friction velocity.

Equation 4.5.2 is well established experimentally for pipes.

The coefficients c_4 and c_5 depend mainly on the shape of the cross sections. Equation 4.5.1a is easier to use than 4.5.2 but c_6 and c_7 depend on the shape of the cross sections and on the roughness ratio $\frac{R}{k_s}$.

For broad channels equation 4.5.2 was given by Keulegan as:

$$\frac{1}{\sqrt{f}} = 1.5 + 2 \log \frac{2R}{k_s} \qquad 4.5.2a$$

This can also be written as:

$$\frac{v_m}{v_*} = 5.66 \log 11.2 \frac{R}{k_s} \qquad 4.5.2b$$

It would certainly be useful if k_s could be estimated with Wolman's sampling method. The difficulty lies in the fact that the energy losses in rivers are only partly the result of grain roughness. A good part of the energy loss is usually due to bars,

bed roughness and meandering.

Of the seven reaches described in this report only the reaches "Quesnel at Lawless Creek" and "Chilko at Henry's Crossing" are so straight and uniform that a correlation between f and ϕ could be expected. The channel of the Chilko River contains a few large boulders (see picture 16), but none could be seen in the Quesnel River (picture 2). Values of k_s have been calculated from equation 4.5.2a for all the river reaches with reasonably uniform flow and for the laboratory test runs 1, 3 and 4, (Table 11). Instead of R, d* has been used throughout.

Two values for the river described in Ref. 14 are also included. This river, the Hasli-Aare, is a gravel river that has been trained between rough rigid banks some 80 years ago. On April 27, 1936, it was at low stage and no bed-load transport took place. On June 13, 1938, it was in full flood with considerable bed load transport.

As it appeared that Φ_{90} , b_{90} and D_{90} gave the best correlation with k_s (note that this confirms the findings of Meyer-Peter, Ref. 19), these values are also listed. They are based on areal grid samples for the rivers of this study (Fig. 10, 16, 29, 38 and 44 and Table 2) and on volumetric surface samples for the laboratory data (Fig. 53 and Fig. 57) and for the Hasli-Aare.

The agreement between k_s and b_{90} or D_{90} appears very good for all the straight uniform channels. This again indicates that approximations of volumetric sieve curves can be obtained by areal grid sampling. The k_s calculated for the Cariboo River at Quesnel Forks is somewhat surprising. The badly defined

slope of this reach might be responsible (Fig. 15). The two k_s values calculated for the Hasli-Aare show that equation 4.5.2a does not hold when the bed is in motion. For the reaches "Taseko River below Taseko Lake" and "Chilko River below Chilko Lake" the fraction of the total energy loss that is spent on grain roughness has been estimated (F). It agrees with the values suggested in Ref. 2.

On Fig. 60 $\frac{V_m}{V_*}$ has been plotted against $\frac{d^*}{k_s}$ for the following data:

(a) The straight and uniform channels of Table 11. k_s is taken as b_{90} or D_{90} respectively.

(b) The two gravel canals described in Ref. 5. k_s is taken as D_{90} and this value appears to be based on a volumetric surface sample. R is used instead of d* for these two points. (Table 13).

(c) The San Luis Valley canals described by Lane in Ref. 20. k_s is here the D_{90} of the material through which the canals were built. It is possibly somewhat smaller than the D_{90} of surface samples but it is the best parameter available. Lane did take surface samples, but not as shown in Fig. 57. (See 5.1 and Table 13).

The curve corresponding to equation 4.5.2b and two straight lines corresponding to equations of the type 4.5.1a are also shown. If a better k_s value had been available for Lane's canals, there would probably be no points significantly above the line of equation 4.5.2b. Some scatter below this line has to be expected, as grain roughness of the channel boundaries is

only one of the causes of energy loss. For $\frac{d^{*}}{k_s}$ ratios of less than 15 an exponential equation of the 4th-root type seems to fit the data best. For greater ratios a line corresponding to Manning's equation seems to fit better. (See also Ref. 17, p.24).

4.6 AREAL SAMPLING WITHOUT A GRID

A few samples were collected by picking all the stones with $b \geqq 0.2$ ft. which were exposed within a certain small area. This corresponds somewhat to the "surface samples" of Ref. 12, 21. Based on the arguments put forward in 4.3.1 such samples should be plotted against frequency by weight, to give something comparable to a volumetric sieve curve. Fig. 39 shows plots by weight and by number. It is interesting to note that the frequency curves of a sample taken from the river bed pavement and of another sample taken from a road cut through the river bank plot almost identically. This sampling technique has two severe handicaps:

1. The lower limit of grain sizes to be considered is arbitrary, yet without such a limit sampling is almost impossible.

2. One of the greatest advantages of the areal sampling with a grid is lost, namely the averaging out in one sample of the material exposed over a considerable area.

CHAPTER 5 THE EQUILIBRIUM OF GRAVEL CHANNELS WITH LOW SEDIMENT CHARGE

5.1 OUTLINE AND ASSUMPTIONS

In this chapter an attempt will be made to find the rules that correlate the stream channel geometry of a special type of gravel channel with discharge and sediment size. River regime in general was briefly discussed in Chapter 1, (See particularly equation 1.3.2.1). Here the following simplifying assumptions will be made:

1. Straight channels with uniform flow.
2. Sediment discharge negligible or zero.
3. Discharge constant with time.
4. Water temperature and sediment properties other than grain size distribution unimportant.

If a channel meets these requirements one can safely assume that the dependent variables shown on the right side of equation 1.3.2.1 are constant with time. Of these variables only the slope and the cross section as defined by width of water surface, depth and area will be of interest here.

The laboratory data meet the above assumptions almost completely, but the seven river reaches described in Chapter 2 only meet them partially. Considerable scatter has to be expected in all plots because of this and because of incomplete field data, rough estimates of dominant discharge and uncertain geomorphological conditions.

The main conclusion derived from Chapter 4, was that grain sizes obtained from grid samples and worked out by number using ϕ or b as linear parameters are very similar to the grain sizes

obtained by sieving volumetric surface samples (which are analysed by weight) of the same areas. It was also concluded that it is preferable to work in terms of the coarser diameters of a given sample (D_{90}, D_{85}, ϕ_{90}, b_{65}, etc.) because in this range agreement between different sampling techniques is likely to be better. These two conclusions will be used extensively in this chapter.

The U.S.B.R. study of gravel canals (Lane, Ref. 20) will be used to fill the wide gap in discharge and grain size between the laboratory and the field data. These canals probably carried a small bed material load. Lane (Ref. 20 and 21) states that occasionally coarse material entered through the head gates.

In the hope of finding some characteristics that are peculiar to gravel rivers with low sediment discharge, a few points for channels with substantial bed load will be included in all the plots. These channels are:

1. Laboratory Test 1, during aggrading (see Table 10).

2. The Aare river near Brienzwiler (Switzerland) as described in Ref. 14. A discharge of 6,720 cfs (2 year flood) will be taken as the dominant discharge and the water surface slope and average cross section of measuring reach 1 (1,500 feet long) will be accepted as representative. At a discharge of 5,000 cfs a bed load charge of 90 ppm was measured.

3. Canals 12 and 13 of the field data of Ref. 5. They are the only gravel canals in this set of observations. They carry a heavy sediment charge of approximately 100 ppm, two thirds of which appear to be bed load.

The hydraulic data of the rivers and canals on which this chapter is based, are listed in Tables 12 and 13. The laboratory data are summarized in Table 10. The grain size distribution for the laboratory tests can be seen on Fig. 53. Lane's bed material samples are comparable to the areal samples without grid (Table 2 and Fig. 39), which the author collected on the reach "Chilko at Henry's Crossing". To make his results comparable with sieve curves of the top layer or with areal grid samples, the author adjusted them by using Figures 38 and 39.

5.2 BRIEF SURVEY OF REGIME EQUATIONS

The concept of regime originated from the maintenance of unlined irrigation canals (Ref. 2, Chapter 2). Here the important independent variables are essentially constant with time, the number of variables is reduced, and the establishment of regime can be observed without great difficulty. From the observation of irrigation canals several competing sets of equations have been developed for the design of new canals (Ref. 2, 5-9). The usual approach is to try and give the width, depth and slope of a canal in terms of water discharge. The great majority of the regime equations make W proportional to $Q^{\frac{1}{2}}$, d or R proportional to $Q^{\frac{1}{3}}$ and S proportional to between $Q^{-\frac{1}{9}}$ and $Q^{-\frac{1}{4}}$. The difference lies in the way in which the proportionality constants are correlated to bed load, sediment discharge and bed and bank material. The disagreement is more pronounced in the form of the relationships than in the actual result, because the canal data from which the various equations were derived are almost identical.

The U. S. Geological Survey has made an extensive investi-

gation of rivers by using data from selected gauging sites (Ref. 10). The following average relations were derived from measurements made at different gauging stations within one drainage basin. Q refers to discharges of equal frequency of exceedance.

$$W \sim Q^{0.5} \qquad 5.2.1$$

$$d \sim Q^{0.4} \qquad 5.2.2$$

$$v_m \sim Q^{0.1} \qquad 5.2.3$$

This seems to indicate that regime equations of the type outlined above can be applied to rivers.

This is not the place to discuss the various sets of regime equations. However, the basic relations which led to Blench's and Lacey's equations (Ref. 2, Ref. 6-9) have to be outlined briefly, because their applicability to the gravel channels of this study will be investigated in paragraph 5.3.

Lacey found that v_m was proportional to $R^{\frac{1}{2}}$ in several Indian canal systems. He therefore defined a silt factor f as

$$f = 0.73 \frac{V_m^2}{R} \qquad 5.2.4$$

The 0.73 was included to make f = 1 for a well known set of canal data. He assumed that f was a function of the grain size of the bed material. As a next step Lacey concluded that $A f^2$ ought to be a function of v_m and finds $A f^2 = 3.8 v_m^5$, which leads to

$$v_m = 0.8 f^{\frac{1}{3}} Q^{\frac{1}{6}} \qquad 5.2.5$$

and

$$P = 2.67 Q^{\frac{1}{2}} \qquad 5.2.6$$

where P stands for "wetted perimeter", $\frac{A}{R}$.

The plots supporting these conclusions are shown in Ref. 9.

In Ref. 8 Lacey deals with the regime slope. On the basis of Indian canal data he concludes that

$$R^{\frac{1}{2}} S = \text{constant} \qquad 5.2.7$$

for a given type of bed material (in Lacey's terms, for constant f). This, together with a flow formula of the fourth root type (see 4.5.1a), leads to his regime slope formula

$$v_m = 16 \, (R^2 S)^{\frac{1}{3}} \qquad 5.2.8$$

This form is supposedly independent of bed and side material. It is supported by a plot of v_m vs $R^2 S$, that includes a wide range of bed materials, starting with gravel rivers of the Swiss Alps and ending with channels on silt.

Blench generalized Lacey's equations to differentiate the effects caused by the sandy beds and by the cohesive sides. He calls $\frac{V_m^2}{d}$ the bed factor. It is supposed to depend on bed material and bed load only. A side factor $\frac{V_m^3}{W}$ is introduced, which is meant to depend on the erosion resistance of the sides and on the tendency of suspended load to deposit on the sides. A certain range of $\frac{V_m^3}{W}$ values is supposedly possible for equilibrium conditions. The dynamical explanation for $\frac{V_m^3}{W}$ given in Ref. 2 applies only to canals with smooth cohesive banks. King's regime slope formula, quoted by Blench, is

$$\frac{V_m^2}{g\,d\,S} = 3.63 \left(\frac{V_m W}{\nu} \right)^{\frac{1}{4}} \qquad 5.2.9$$

It is based on a plot of $\frac{V_m^2}{g\,d\,S}$ vs. $\frac{V_m W}{\nu}$ for Indian canals with steep cohesive banks and sand beds moving in dune formation.

For constant bed and side factor Blench's regime formulas satisfy Lacey's equation 5.2.7 in terms of d instead of R.

When Blench's and Lacey's equations are written as design equations to give R or d, W or P and S as functions of Q, silt factor, bed factor and side factor respectively, the exponent of Q is found to be the same in corresponding equations.

Lane (Ref. 20 and 21 approached the design of unlined

irrigation canals with the idea that a canal would be stable as long as the shear force exerted by the flowing water on the boundaries did not exceed a certain limiting value. This critical or limiting shear force is considered to depend on the diameter of the grains and on the angle at which the material is lying. In the gravel range Lane's critical tractive force curve is well supported by field data.

The tractive force concept cannot be reconciled with the regime formulas of Lacey or Blench, since they stipulate that for a given bed and side material

$$R^{\frac{1}{2}} S = \text{constant} \qquad 5.2.7$$

while the tractive force concept leads to

$$R\ S = \text{constant} \qquad 5.2.10$$

for a broad channel.

5.3 EXAMINATION OF THE DATA

5.3.1 Regime Formulas of Lacey and Blench. The applicability of the regime formulas, outlined in paragraph 5.2, to a certain set of regime data can be tested in various ways. Since correlations between the grain size of the bed material and $\frac{V_m^2}{d}$ and $R^{\frac{1}{2}}$ S are essential for Lacey's and Blench's formulas, plots of grain size versus these parameters have been prepared using all the channels mentioned under 5.1. Grain sizes of several different frequencies ("percent finer than") were used, but none of the plots showed any functional relationship. The use of $\frac{V_m^2}{d^*}$ instead of $\frac{V_m^2}{d}$ did not produce a correlation either. Figures 61 and 62 show the plots of $d^{*\frac{1}{2}}$ S and $\frac{V_m^2}{d}$ against the "65 percent finer than" grain sizes. On the basis of such plots one has to conclude that regime formulas involving silt factor, bed factor or

side factor are not applicable to gravel channels of the type studied here.

Since Lacey's width formula (equation 5.2.6) does not contain the silt factor, an attempt was made to verify it. Fig. 63 shows a plot of Q vs W_s . The correlation is remarkable. The best fit line has the equation

$$W_s = 1.95 Q^{\frac{1}{2}} \qquad 5.3.1.1$$

Simple explanations can be put forward for all the points that fall considerably below the best fit line. The Hasli-Aare was artificially squeezed between rigid banks to induce it to lower its bed. The Thompson River is deeply incised and a few rock outcrops could be seen along its sides. Only one mile below the measuring reach it widens to approximately 550 feet.

Scatter above the fitting line will be examined in 5.3.3.

5.3.2 Tractive Force. The tractive force concept was tested by plotting grain size vs. γd^*S. Fairly good correlations were obtained for several grain size frequencies (percent finer than). On Fig. 65 γd^*S is plotted against the "50% finer than" grain sizes. The fitting line of this plot has the slope of one upon one, which corresponds to Lane's findings. The equation of this line is

$$D_{50} = 0.38 \gamma d^*S \qquad 5.3.2.1$$

The best correlation was obtained by plotting the "90% finer than" grain sizes against γd^*S, as shown on Fig. 64. The equation of this fitting line is

$$D = 0.7 (\gamma d^*S)^{\frac{4}{3}} \qquad 5.3.2.2$$

The scatter on both plots is considerable but one should keep in mind that it is impossible to characterize a natural channel pave-

ment with just one grain size parameter.

The following interesting conclusion can be drawn from Fig. 64 and 65:

The points that corresponds to channels with considerable bed load transport plot on both sides of the fitting line. This throws some doubt on the use of tractive force as a transport criterion. (This has been done by Meyer-Peter in Ref. 19.) From results derived from the laboratory work, the author believes that tractive force is a valid erosion criterion for paved, straight and uniform gravel channels. If d*S increases beyond a critical value the bed starts moving. Once the bed is fully in motion the bed pavement disappears and the apparent roughness of the channel decreases. With it the tractive force may decrease to a lower value than the original critical one. The above only applies to natural gravels with a wide range of grain sizes.

5.3.3 <u>The Uniqueness of the Equilibrium of Gravel Channels.</u>

1. A few points on Fig. 63 plot considerably above the fitting line of equation 5. This seems to indicate that the width of gravel channels in low transport equilibrium has to meet only a certain minimum condition (given by equation 5.3.1.1). Rivers with considerable suspended load build banks out of suspended material and keep their width at a value that is delicately adjusted to discharge and to sediment load.

The rivers of this study however, have very little suspended load and are therefore unable to build banks. Their width seems to fit equation 5.3.1.1 closely, whenever the present-day river is responsible for its channel. If the channel contained a

larger river at some earlier time, as appears to be the case at the Chilko Lake outlet, the width will not adjust to the new discharge. The same naturally applies to gravel canals that have been built wider than necessary. As a design equation 5.3.1.1 should probably be written as

$$W_s \geq 1.95\, Q^{\frac{1}{2}} \qquad 5.3.3.1$$

2. The Chilko River below Chilko Lake also plots considerably above the fitting lines of Fig. 64 and Fig. 65. This indicates that equations 5.3.2.1 and 2 should have the equal sign replaced by an "equal or greater than" sign ($\geqq$), in order to be applicable to all gravel channels. The Chilko Lake outlet appears to be over-stable (the channel could take a considerably larger discharge without being eroded) and not in regime.

3. The main aim of two of the laboratory tests was to investigate the uniqueness of slope and depth for a given discharge, bed material and width. The equilibrium was approached by building up a bed in Test 1b and by degrading an oversteepened bed in Test 3. The resulting depth and slope are very close but not identical. In channels with fixed width, the way in which equilibrium is reached therefore appears to have only a minor influence on the final no-transport equilibrium.

5.4 REGIME EQUATIONS FOR GRAVEL CHANNELS WITH SMALL SEDIMENT LOAD.

With the simplifying assumptions made for this chapter (see 5.1) the general regime function (1.3.2.1) reduces to:

$$W_s,\ d^*,\ S,\ v_m = \text{fns}\ (Q,\ \text{bed material}) \qquad 5.4.1$$

or to:

$$W,\ d,\ S,\ v_m = \text{fns}\ (Q,\ \text{bed material}) \qquad 5.4.1a$$

The author prefers equation 5.4.1 because W_s is easily measured in the field. It will be assumed that the bed material can be characterized with one linear parameter like ϕ_{90}, D_{50}, etc.

Four independent equations are needed to solve 5.4.1. So far, three are available; namely the equation of continuity,

$$Q = v_m W_s d^* \qquad 5.4.2$$

the width equation 5.3.1.1, and the tractive force equations 5.3.2.1 or 5.3.2.2.

When the flow of water through a gravel channel carries practically no bed load it should be possible to treat it as flow in rigid boundaries by applying the friction formulas discussed in 4.5.

Rivers and canals flowing through sand form ripples and dunes on their beds. The channel roughness therefore depends on the flow conditions. This is the reason why the flow formulas of Lacey and Blench (5.2.8 and 5.2.9) differ from the rigid boundary formulas mentioned in 4.5.

In the field and in the laboratory no ripples or dunes could be detected. Lane, who was able to inspect his canals when they were drained, could not see any either (Ref. 20). This fact, combined with the findings of paragraph 4.5 (particularly Fig. 60), support the conclusion that a rigid boundary flow formula with the grain size as roughness height should be used as a regime equation for gravel channels in low transport equilibrium. An exponential flow formula will be used here because all the other regime equations are also exponential. The fourth root equation

$$v_m = 6.6 \left(\frac{d^*}{k_s} \right)^{\frac{1}{4}} \sqrt{g\, d^* S} \qquad 5.4.3$$

appears to be the best fit to the points of Fig. 60. To test

the assumptions about tractive force and fourth root flow formula one can combine the two by replacing k_s in 5.4.3 with D_{50} or D_{90} of equations 5.3.2.1 or 5.3.2.2 respectively.

Using 5.3.2.1 gives

$$v_m \sim (d^{*2}\ S)^{\frac{1}{4}} \qquad 5.4.4$$

and 5.3.2.2 leads to

$$v_m \sim (d^{*2.5}\ S)^{\frac{1}{6}} \qquad 5.4.5$$

These two equations have been tested on Fig. 66 and 67. Equation 5.4.5 appears to be better supported by the available evidence. From now on the fitting line

$$v_m = 10(d^{*2.5}\ S)^{\frac{1}{6}} \qquad 5.4.5a$$

will be used instead of 5.3.2.2.

The points that do not fall near this fitting line on Fig. 66 correspond to channels with considerable bed load charge or to lake outlets which are probably over stable (Chilko and Cariboo). Fig. 66 therefore appears to be a useful tool for testing the no-transport equilibrium of a gravel channel.

The four equations which are necessary to solve the simple regime problem of 5.4.1 are now available. They are:

$$Q = v_m\ W_s\ d^* \qquad 5.4.2$$

$$W_s = 1.95\ Q^{\frac{1}{2}} \qquad 5.3.1.1$$

$$v_m = 6.6\left(\frac{d^*}{k_s}\right)^{\frac{1}{4}} \sqrt{g\ d^* S} \qquad 5.4.3$$

$$v_m = 10(d^{*2.5}\ S)^{\frac{1}{6}} \qquad 5.4.5a$$

If they are rewritten to give the dependent variables W_s, d^*, S and v_m in terms of Q and k_s, the following equations result:

$$W_s = 1.95\ Q^{0.5} \qquad 5.3.1.1$$

$$d^* = 0.159\ Q^{0.4}\ k_s^{-0.1} \qquad 5.4.6$$

$$S = 0.1172\ Q^{-0.4}\ k_s^{0.85} \qquad 5.4.7$$

$$v_m = 3.22\ Q^{0.1}\ k_s^{0.1} \qquad 5.4.8$$

k_s is the Nikuradse grain size roughness measured in feet. According to the conclusions of 4.5 it can be replaced by D_{90}, ϕ_{90} or b_{90}.

The exponents of Q are equal to those found by the U.S.G.S. (Ref. 10 and equations 5.2.1, 5.2.2, and 5.2.3).

If Q is replaced in 5.4.6 by using 5.4.7 one obtains a new fitting line for Fig. 64. This equation is

$$k_s = 0.818\ (\gamma\, d^* S)^{\frac{4}{3}}$$

which is not very different from the original one.

The above equations do not apply to channels with fixed width (flume experiments, trained rivers, overly wide canals). For such applications 5.4.6 and 5.4.7 have to be replaced by

$$d^* = 0.268\ Q^{0.8}\ k_s^{-0.1}\ W_s^{-0.8} \qquad 5.4.9$$

$$S = 0.071\ Q^{-0.8}\ k_s^{0.85}\ W_s^{0.8} \qquad 5.4.10$$

These last two equations could be checked against Meyer-Peter's gravel transport experiments (unpublished) using only the few ones with extremely low transport rates. The agreement is reasonably good particularly if one takes into consideration that Meyer-Peter's experiments were made by adjusting depth and discharge to obtain a certain transport rate and a certain water surface slope. The sequence of cause and effect in a river is evidently different.

5.5 SOME DIFFERENCES BETWEEN SAND AND GRAVEL RIVERS

The regime formulas that have been derived in 5.4 differ appreciably from the regime formulas of Lacey and of Blench. This indicates a distinct difference between the mechanisms of sand and gravel transport, as there can be little doubt that these last formulas represent the regime of irrigation canals with sandy beds quite well (Ref. 5-9). The author's time and

data are insufficient for a thorough study of the reasons behind this difference but he would like to point out a few pieces of evidence that seem to support his belief that gravel and sand rivers cannot be expected to obey the same laws.

5.5.1 <u>The Pavement of Gravel Channels</u>. Sundborg in Ref. 22, points out how the relatively rare flume experiments using material with a wide range of grain sizes have come to different conclusions depending on what type of material was used. Experiments with materials predominantly in the sand range have shown that the coarsest grains, if not coarser than approximately 6 mm, tend to move first and most easily by sliding across the sandy bed. Experiments in the gravel range have indicated that the finer a grain is the more likely it is to move.

The latter implies that a gravel river must have its bed paved with the coarsest material available, if it is in a low transport equilibrium. An easy way to see this is by imagining a gravel river with a bed that is not paved and that consequently consists of a wide range of grain sizes. Such a river could only have low transport rates by being overstable (unable to move a major portion of the bed material) and therefore not in regime. If it were in regime the transport rates of the smaller grain size fractions would be high. The fact that the rivers mentioned in Chapter 2 and the laboratory tests showed distinct gravel pavements is an indirect proof of this second part of Sundborg's statement. The first section of the statement is supported somewhat by observations made during the aggrading part of test 1. Sandy patches formed on the bed. Gravel grains moved across such patches with great ease. They hardly ever came to rest on a sandy patch.

A second implication of Sundborg's statement is that river reaches with bed material predominantly in the 2-4 mm range should not exist. This material would move with greater facility than coarser grains through the gravel reaches of a river and more easily than sand in the sandy reaches. Because of this ease in moving along the river, accumulation of 2-4 mm material is unlikely to occur. Kuiper (Ref. 23) and Halferdahl (L. B. Halferdahl, Geologist with the Alberta Research Council, oral communication) have investigated several of the larger rivers of the Canadian Prairies. They both noticed abrupt changes from gravel to sand.

For the same reasons the 2-4 mm grain sizes should be deficient in river deposits in the areas where the change from gravel to sand occurs. The gravel samples taken in the Edmonton area seem to confirm this (Fig. 51).

5.5.2 Bed Roughnesses. The second important difference between sand and gravel transport, the absence of ripples and dunes in certain gravel channels, has already been mentioned in 5.4. Ref. 15 describes hydraulic transport experiments with gravel of a wide grain size range. At high transport rates dunes formed, but at low transport rates the bed stayed flat and developed a pavement. Whether dunes form seems to depend on the roughness ratio $\frac{d^*}{k_s}$ and on the transport rate.

Two of the main formulas of paragraph 5.4 (tractive force 5.3.2.1 and the fourth root flow formula 5.4.3 with grain size as roughness) definitely do not apply to channels with ripples or dunes on the bed. It is therefore unlikely that the regime of the gravel rivers that do not form ripples or dunes on their beds

can be described by formulas having the same form as equations 5.4.6 to 5.4.10.

5.6 THE DOWNSTREAM EFFECTS OF A DAM

Rivers usually lose their sediment load in the ponds created by major dams. Downstream of such a dam the rivers have to adjust to the enforced low sediment transport rates and possibly also to a lower dominant discharge.

Sand rivers adjust by changing their channel cross section and their slope (Ref. 2, 10). Gravel rivers have the added possibility of changing the characteristics of the bed material by developing a pavement. Instead of lowering the slope such a river will degrade, by picking the finer grains off the bed, until the bed pavement is coarse enough to resist further degrading (Ref. 15 is a study of the formation of bed pavements). In terms of the equations developed in 5.4 the river will degrade until k_5 meets the requirements of equation 5.4.10 for the existing width and slope.

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APPENDIX A

SYMBOLS AND ABBREVIATIONS

Symbols

a	large	axis of a stone
b	intermediate	
c	small	

ϕ equivalent spherical diameter

The indices of a, b, c and ϕ (b_{70} , ϕ_{50}) mean, unless otherwise specified, "percent finer than" based on areal grid samples worked out by number.

D_f grain size based on sieve curve of volumetric sample worked out by weight. The index f stands for "percent finer than".

A area of channel cross section

d depth in the central parts of a channel

d* $\frac{A}{W_s}$

W_s water surface width

W $\frac{A}{d}$

P wetted perimeter

R hydraulic radius

Q discharge

v velocity

v_m average velocity of a cross section

v_s surface velocity

S slope

ν	kinematic viscosity	of water
γ	specific weight	

g	acceleration of gravity
f	friction factor $\frac{8gRS}{V_m^2}$
k_s	Nikuradse grain size roughness
°F	degree Fahrenheit
V	volume

All dimensional equations are in the foot-pound-second system.

Abbreviations

ASCE	American Society of Civil Engineers
IAHR	International Association for Hydraulic Research
ICE	Institution of Civil Engineers (G.B.)
IPSFC	International Pacific Salmon Fisheries Commission
U.S.G.S.	U. S. Geological Survey
U.S.B.R.	U. S. Bureau of Reclamation
Ref.	reference(s)
Fig.	figure(s)
Proc.	proceedings
Trans.	transactions
st.	station
l.b.	left bank
r.b.	right bank
M.C.	main channel
sq. m.	square mile

APPENDIX B, TABLES

TABLE 1

COMPARISON BETWEEN CALCULATED AND MEASURED WEIGHT OF STONES

River	No. of Stones	Weight (grams)	Calculated Weight (grams)
Cariboo at Q.F.	16	9231	9560
Chilko at Bridge	8	1227	1180
Taseko	5	10,440	9270
Taseko	2	14,700	12,400
Taseko	9	2537	2310
Taseko	10	3775	3420
Aare (Ref.14)	1	63,000	43,500
	1	40,000	43,500
	1	10,000	6,500
	1	15,000	8,700
	1	85,000	78,000
	1	17,000	11,800
	1	15,000	17,400
	1	9,000	7,200
	1	60,000	67,400
	1	17,000	23,500
	1	9,000	9,100
	1	55,000	82,600
	1	12,000	12,400
	1	3,000	1,700
Aare, (Ref.14), total sample	14	410,000	343,300

TABLE 2

SUMMARY OF GRAVEL SAMPLES

Abbreviations:

T sampled above water along a survey tape.
W sampled below water by random walking.
A sampled by picking all the stones with b greater than 0.2 ft, exposed within a certain area.
ϕ equivalent spherical diameter in inches.
b intermediate axis in inches.
The indices of ϕ and b are "percent by number finer than".

Sample No.	Date	Location	Stones in Sample	Method of Sampling	ϕ_{50}	b_{50}	ϕ_{65}	b_{85}	$\frac{\phi_{85}}{\phi_{15}}$	$\frac{b_{85}}{b_{15}}$
		(a) QUESNEL RIVER AT LAWLESS CREEK								
1	Sept.21	St. 3, r.b.	76	T	8.0	8.9	9.2	10.6	2.72	2.7
2	Sept.20	Between st.3 and 4,r.b. at July waterline.	50	T	6.3	7.2	7.8	9.1	3.10	2.90
3	Sept.20	St.5, r.b. at waterline.	78	T	2.6					
4	July 3	In old channel near st.3,r.b.	79	T	5.0					
5	July 4	In old channel near st.5, r.b.	66	T	7.2	8.4	9.0	10.7	2.82	2.90
		(b) CARIBOO RIVER AT QUESNEL FORKS								
1	Sept.19	St.2,r.b. along water-line	61	T	7.5	8.4	9.5	10.7	3.50	3.56

TABLE 2 (cont'd)

Sample No.	Date	Location	Stones in Sample	Method of Sampling	ϕ_{50}	b_{50}	ϕ_{65}	b_{65}	$\frac{\phi_{84}}{\phi_{16}}$	$\frac{b_{84}}{b_{16}}$
		(b) CARIBOO RIVER AT QUESNEL FORKS (cont'd)								
2	Sept.18	On bar in centre of river near st. 2.	31	T	9.3	11.0	12.0	13.2	2.49	2.46
3	Sept.18	Along water at st. 3, 1.b.	43	T	8.0					
4	Sept. 17	At st.2, r.b.,	59	T	5					
4a		along high water mark	33		4.3					
4b		along water line	28		6.4					
5	Sept.20	Along water up from st.3,1.b.	56	T	9.2	10.0	10.8	12.1	2.54	2.66
		(c) CARIBOO RIVER AT CARIBOO LAKE								
1	Sept.23	St.4, rb. along waterline.	55	T	2.9					
2	Sept.23	St.3,r.b. along waterline	51	T	3.7					
3	Sept.24	St.3,1.b. along waterline	59	T	4	5.0	5.4	6.7	4.85	4.60
4	Sept.24	St.5,1.b. along waterline	59	T	6.1	7.7	7.5	9.0	3.06	2.85
5	Sept.25	On bar near St.3	20	W	7.5	8.0	8.2	8.8	1.57	1.64

TABLE 2 (cont'd)

Sample No.	Date	Location	Stones in Sample	Method of Sampling	ϕ_{50}	b_{50}	ϕ_{65}	b_{65}	$\frac{\phi_{84}}{\phi_{16}}$	$\frac{b_{84}}{b_{16}}$
		(d) TASEKO RIVER BELOW TASEKO LAKES								
1	Aug. 14	On steep bank, 100' below st.3, r.b.	57	T	1.85					
2	Aug. 19	At st. 0, r.b. about ½ ft. above water	53	T	5.6					
3	Aug. 21	On bar between st.3 and st.4, l.b.	47	T	3.4					
4	Aug. 21	From 2 ft. depth at st.2, l.b.	6	W	3.5					
5	Sept. 8	1-2 ft. depth near st.2, r.b.	26	W	5.25	6.36	6.5	7.85	2.90	2.62
6	Sept. 8	Along water between st.1 and st. 2, r.b.	66	T	5.3	5.9	6.5	7.2	3.7	3.1
		(e) CHILKO RIVER AT HENRY'S CROSSING								
1	Sept.11	St.1, r.b. under 1 ft. of water	58	A	3.95					
2	Sept.11	70' upstream of st. 4, r.b. from under 1.5 ft of water.	15	W	6.7	6.3	7.6	7.3	1.95	2.2

TABLE 2 (cont'd)

Sample No.	Date	Location	Stones in Sample	Method of Sampling	ϕ_{50}	b_{50}	ϕ_{65}	b_{65}	$\frac{\phi_{84}}{\phi_{16}}$	$\frac{b_{84}}{b_{16}}$
		(e) CHILKO RIVER AT HENRY'S CROSSING (cont'd)								
3	Sept. 4	On bar upstream of bridge, under 1-2 ft. of water.	30	W	5.3	5.6	5.9	6.3	2.05	1.83
4	Sept.12	In side channel near st.2, l.b.	17	W	4.3					
5	Sept.12	St.3, r.b. from under 1.5 ft. of water.	8	W	5.5					
6	Sept.12	St.4 along water-line.	43	T	4.8	4.9	6.1	5.9	3.68	3.04
7	July 20	Roadcut at bridge.	66	A	3.6					
8	Sept.14	St.2,r.b. on bank just above bankfull stage	60	A	3.4					
		(f) CHILKO RIVER AT THE OUTLET OF CHILKO LAKE								
1	Sept.13	Between st. 2 and 1, 70 ft. out from r.b.	31	W	3.45					
2	July 23	St.2,75 ft. out from r.b.	50	W	3.2	3.4	3.9	4.2	3.38	3.06
3	July 23	St.2, l.b., 15 ft. out, depth 2-2.5 ft.	73	W	2.9	3.0	3.4	3.5	2.52	2.38

TABLE 2 (cont'd)

Sample No.	Date	Location	Stones in Sample	Method of Sampling	ϕ_{50}	b_{50}	ϕ_{65}	b_{65}	$\frac{\phi_{84}}{\phi_{16}}$	$\frac{b_{84}}{b_{16}}$
		(g) THOMPSON RIVER BELOW KAMLOOPS LAKE								
1	Sept.28	On bar at st.10 r.b.	80	T	6.3	7.0	7.2	7.9	2.21	2.14

TABLE 3

QUESNEL RIVER AT LAWLESS CREEK

- Discharges on days when hydraulic measurements were made:

July 2, 1962	14,800 cfs
Sept.20, 1962	5,340 cfs

- Water temperature at high stage: 47° F

- Slope measurements:

	Date	Reach	Slope
Water surface	July	1-6	$6.53.10^{-3}$
	September		$5.68.10^{-3}$
Energy line	July	1-5	$6.33.10^{-3}$
	September		$5.16.10^{-3}$

- Summary of cross section measurements (July 2, 62) :

Stn. No.	Width of Water Surface (ft)	Estimated Depth (ft)	Cross Section Area (ft^2)	V_m ($ft\ s^{-1}$)	Measured V_s ($ft\ s^{-1}$)
1	202	8.1	1340	11	
2	200	8.5	1360	10.9	12.3
3	216	7.5	1240	11.9	
4	150	7.8	1020	11.6	14.6 *
5	230	5.6	1190	12	

* M.C. carrying approx. 80% of total discharge

- Average section at dominant discharge:

Width of water surface	200	ft
Depth	8.0	ft
Area	1300	ft^2
Dominant discharge	14,800	cfs

- Flood Frequency:

Recurrence Interval (years)	Discharge (cfs)
2	13,400
5	16,700
10	18,600
20	20,300

TABLE 4

THE CARIBOO RIVER AT QUESNEL FORKS

- Discharges on days when hydraulic measurements were made:

June 29,1962	12,000 cfs
July 1 ,1962	10,500 cfs
Sept. 17,1962	2,810 cfs

- Water temperature at high stage: 46° - 48° F

- Slope measurements:

	Date	Reach	Slope
Energy Line	June 29	1-4	$4.19.10^{-3}$
		1-3	$4,62.10^{-3}$
Water Surface	June 29	1-5	$3.71.10^{-3}$
		1-3	$4.81.10^{-3}$

- Summary of cross section measurements: (June 29, 62)

Stn. No.	Width of Water Surface (ft)	Depth (ft)	Cross Section Area (ft^2)	$\bar{v}_m$ ($ft\ s^{-1}$)	Measured $\bar{v}_s$ ($ft\ s^{-1}$)
1	165	9	1260	9.5	11.5
2	275	6.5	1150	10.4	
3	192	8	1170	10.3	
4	226	8.5	1544	7.8	11.2

- Average section at dominant discharge:

Width of water surface	190	ft
Area	1250	ft^2
Depth	8.5	ft
Dominant discharge	12,000	cfs

- Flood frequency:

Recurrence Interval (years	Discharge (cfs)	Depth of Average Section (ft)
2	12,700	8.5
5	15,000	9.3
10	17,000	
20	18,500	

TABLE 5

THE CARIBOO RIVER AT THE OUTLET OF CARIBOO LAKE

- Discharges on days when hydraulic measurements were made:

July 8, 1962	6560	cfs
July 10, 1962	6710	cfs
Sept. 23, 1962	2180	cfs

- Water temperature at high stage: 46° - 48° F

- Slope measurements:

	Date	Reach	Slope
Energy line	July 10	2-6	$1.64.10^{-3}$
	Sept. 23	2-6	$2.07.10^{-3}$
Water surface	July 8	2-7	$1.92.10^{-3}$
	July 10	2-7	$1.92.10^{-3}$
	Sept. 23	2-7	2.12.10

- Highest discharge in 1962: 10,900 cfs at a gauge reading 2.5 ft. higher than the reading of July 10.

- Summary of cross section measurements (July 10, 62) :

Stn. No.	Width of Water Surface (ft)	Depth of Main Channel (ft)	Section Area (ft^2)	d* (ft)	$\bar{v}_m$ ($ft\ s^{-1}$)	Measured $\bar{v}_s$ (ft s)
2	233	11.9	1845	7.9	3.65	
3	275	9.5	1560	5.7	4.3	
4	274	9	1563	5.7	4.3	7
5	247	7	1218	5.0	5.5	
6	280	7.5	1680	6.0	4.0	7.7

- Average section at dominant discharge:

Width of water surface	260	ft
Area	2800	ft^2
Depth	13.0	ft
Dominant discharge	17,000	cfs
Bankfull discharge	8,000	cfs

- Flood frequency:

Recurrence Interval (years)	Discharge (cfs)	Estimated Depth of Average Section (ft)
2	11,300	10.1
5	13,200	11.1
10	15,000	12.1
20	16,300	12.8
50	17,600	13.9
100	20,600	14.9

TABLE 6

THE TASEKO RIVER AT THE OUTLET OF TASEKO LAKE

- Discharge on days when hydraulic measurements were made:

Aug.	13	3500	cfs
Aug.	15	3330	cfs
Aug.	17	3330	cfs
Aug.	19	3170	cfs
Aug.	21	3550	cfs
Aug.	22	3560	cfs
Sept.	8	2500	cfs

- Water temperature at high stage: approximately 48° F

- Water surface slope:

Date	Reach	Slope
Aug. 13, 62	1-5	$2.87.10^{-3}$
Aug. 21, 62	1-5	$2.91.10^{-3}$

- Channel dimensions at station 2, Aug. 21, 62

Width of water surface	135	ft
Depth	4.8	ft
Area	528	ft^2
V_m	6.7	$ft\,s^{-1}$
Measured V_s	8	$ft\,s^{-1}$

- Average channel dimensions at dominant discharge:

Width of water surface	150	ft
Area	800	ft^2
Depth	6.2	ft
V_m	8.0	ft
Dominant discharge	6400	cfs

- Flood frequency:

Recurrence Interval (years)	Discharge (cfs)
2	5400
5	6100
10	6500
20	6900

TABLE 7

CHILKO RIVER AT HENRY'S CROSSING

- Discharge on days when hydraulic measurements were made:

Aug.	18	1959	3500	cfs
July	16	1962	3420	cfs
July	18	1962	3630	cfs
Sept.	11	1962	2540	cfs.

- Water temperature at high stage: 48° - 50° F

- Water surface slope:

Date	Reach	Slope
July 18, 62	0-5	$5.03.10^{-3}$
Sept.11, 62	0-5	$5.08.10^{-3}$

- Summary of cross section measurements: (July 18, 62)

Stn. No.	Width of Water Surface (ft)	Estimated Depth (ft)	Cross Section Area (ft^2)	$\bar{v}_m$ ($ft\ s^{-1}$)	Measured $\bar{v}_s$ ($ft\ s^{-1}$)
0	165	4.1	575	6.3	9.6
1	135	4.2	460	7.9	
2 **	147	4.1	525	6.2	
3	140	4.7	565	6.4	7.3
4	155	4.2	545	6.8	8.5
5 *	137	4.7	600	5.9	

* IPSFC section
** Assuming 90% of discharge is in main channel

- Average section at dominant discharge:

Width of water surface	150	ft
Depth	5.5	ft
Area	690	ft^2
Dominant discharge	5600	cfs

- Flood frequency:

Recurrence Interval (years)	Discharge (cfs)
2	4800
5	5600
10	6100
20	6500

TABLE 8

CHILKO RIVER AT THE OUTLET OF CHILKO LAKE

- Discharge on days when hydraulic measurements were made:

July 24,	1948	4600 cfs
July 22,	1962	3340 cfs
Sept 13,	1962	2370 cfs

- Water temperature at high stage: 48° - 50° F

- Water surface slope:

Date	Reach	Slope
July 22, 62	1-3	$0.92.10^{-3}$
Sept.13, 62	1-3	$0.94.10^{-3}$
* July 22, 48	3700 ft	$1.19.10^{-3}$

* IPSFC

- Summary of cross section measurements: (July 22, 1962)

Stn. No.	Width of Water Surface (ft)	Depth of Main Channel (ft)	Area of Cross Section (ft^2)	Area of Main Channel Cross Section (ft^2)	v_m Main Channel* ($ft\ s^{-1}$)
1	272	5.2	1138	988	3.0
2	390	5.1	1192	735	3.6
3	263	5.2	1026	812	3.3

* assuming 80-90% of discharge is in main channel.

- Average section at dominant discharge:

Width of water surface	270	ft
Depth of main channel	7.5	ft
Area of main channel	1413 *	ft^2
Total area	1760	ft^2
Dominant discharge	7000	cfs
Bankfull discharge	5000	cfs

* carrying 80% of discharge.

- Flood frequency:

Recurrence Interval (years)	Discharge (cfs)
2	4800
5	5600
10	6100
20	6500
50	7000

TABLE 9

THE THOMPSON RIVER BELOW KAMLOOPS LAKE

- Discharge on days when hydraulic measurements were made:

Aug. 17, 1961	23,700 cfs
Aug. 18, 1961	24,000 cfs
Sept. 28, 1962	18,600 cfs

- Water temperature at high stage: 48° - 50° F

- Water surface slope:

  Aug.17-18,1961, from section 7 to section 17: $0.72.10^{-3}$

- Summary of estimated cross section dimensions at dominant discharge:

Station Number	Width of Water Surface (ft)	Area (1000 ft^2)	Average Depth (ft)	Maximum Depth (ft)	v_m ($ft\ s^{-1}$)
7	487	12.4	32.2	39.3	10.9
8	442	10.1	29.1	31.7	13.3
9	405	11.0	46.5	49	12.2
10	498	11.6	31.8	42.2	11.6
11	472	12.6	39	43	10.7
12	425	10.5	33.2	37	12.9
13	400	11.0	34.2	44	12.2
14	545	14.4	33.2	38.5	9.4
15	490	12.3	31	33.5	11
16	340	11.2	40	56	12
17	610	18.2	35.2	45	7.4

- Average section at dominant discharge:

Width of water surface	450	ft
Depth	34	ft
Area	11,500	ft^2
Dominant discharge	135,000	cfs
Bankfull discharge	110,000	cfs

- Flood frequency:

Return Period (years)	Discharge (1000 cfs)
2	94
5	110
10	119
20	126
50	135

TABLE 10

SUMMARY OF LABORATORY EXPERIMENTS

Test	Dis-charge (cfs)	Average Water Surface Slope (%)	Average Depth (ft)	Width (ft)	v_m ($ft\ s^{-1}$)	Measured v_s ($ft\ s^{-1}$)
1 agr.	1	1.31	0.10	3.4	2.80	2.92
1 degr.	1	1.20	0.14	3.4	2.18	2.85
2	1	0.99	0.22	2.6	2.10	3.33
3	1	1.24	0.15	3.0	2.27	
4	2.25	0.625	0.26	3.4	2.57	3.34

TABLE 11

EQUIVALENT SAND GRAIN ROUGHNESS

Channel	Date	k_s * (ft)	Φ_{10} (ft)	b_{10} (ft)	D_{90} (ft)	F
Quesnel at	July 2,62	1.24	1.08	1.25		
Lawless Creek	Sept.20,62	1.35				
Chilko at	July 18,62	1.15	0.83	0.83		
Henry's Crossing	Sept.11,62	1.17				
Laboratory	Test 1	0.031			0.033	
	Test 2	0.037				
	Test 3	0.030				
Hasli Aare at	Apr.27,36	0.61			0.59	
Brienzwiler	June 13,38	1.50				
Cariboo at Quesnel forks	June 29,62	1.15	1.4	1.55		
Taseko	Aug.21,62	3.18	0.95	1.00		0.47
Chilko at Chilko Lake	July 22,62	1.55	0.55	0.59		0.62

* k_s is calculated from equation 4.5.2a, using d* instead of R.

TABLE 12

HYDRAULIC DATA OF GRAVEL RIVERS AT DOMINANT DISCHARGE

River Reach	Q cfs	W_s ft	d ft	d* ft	A ft^2	v_m $ft\,s^{-1}$	S $.10^{-3}$	Φ_{65} in	Φ_{50} in	b_{90} in	b_{50} in
Chilko River at Henry's Crossing	5,600	150	5.5	4.6	690	8.1	5.03	6.4	5.5	10.0	5.6
Taseko River below Taseko Lake	6,400	150	6.2	5.3	800	8.0	2.9	6.5	5.3	12.0	5.9
Chilko River at the Outlet of Chilko Lake	7,000	270	7.5	6.5	1,760	4.0	0.92	4.7	4.0	7.0	4.5
Cariboo River at Quesnel Forks	12,000	190	8.5	6.6	1,250	9.6	4.19	11.0	9.5	18.5	10.5
Quesnel River at Lawless Creek	14,800	200	8	6.5	1,300	11.4	6.33	9.2	8.0	15.0	8.5
Cariboo River at the Outlet of Cariboo Lake	17,000	260	13	10.8	2,800	6.1	1.92	7.6	6.2	13.0	7.7
Thompson River below Kamloops Lake	135,000	450	34	25.6	11,500	11.8	0.72	7.3	6.2	10.5	7.0
Hasli-Aare near Brienzwiler (Ref. 14)	6,720	79	13.1	9.8	785	8.54	2.00	D_{65} 4.7		D_{90} 7.1	D_{50} 3.0

TABLE 13

HYDRAULIC DATA OF GRAVEL CANALS AT FULL SUPPLY

Canal		Q cfs	W_s ft	d ft	d* ft	A ft^2	v_m $ft\ s^{-1}$	S $.10^{-3}$	(1) D_{90} in	(3) D_{65} in	(3) D_{50} in
Lane (Ref. 20)											
	#1	1500	73	4.87	3.50	255	5.88	2.80	5.0	2.69	2.32
"	#4	768	48	3.11	2.46	118	6.53	3.59	2.55	2.49	2.15
"	#5	448	40	2.5	1.95	77	5.82	3.68	2.65	1.78	1.51
"	#6	159	21.7	1.88	1.60	34.6	4.59	2.95	2.20	1.38	1.20
"	#7	95.6	15.9	1.73	1.38	21.9	4.36	2.90	1.30	1.35	1.15
"	#8	46	19.2	0.96	0.80	15.3	3.00	3.16	1.45	1.28	1.10
"	#10	16.6	11.1	0.60	0.515	5.7	2.90	9.65	2.50	2.1	1.80
"	#11	203	32.3	1.88	1.62	52.4	3.88	2.35	1.70	1.58	1.36
"	#12	128	21.9	1.77	1.46	32.0	4.00	2.43	1.45	1.12	0.96
"	#14	110	21.4	2.00	1.56	33.4	3.29	1.39	0.66	0.66	0.56
"	#15	477	39.4	3.05	2.50	99	4.84	1.99	2.45	1.64	1.42
"	#17	531	41	2.60	2.35	96	5.51	2.74	1.90	1.25	1.08
Simons (Ref.5)									(2) D_{85}	(4)	(2)
	#12	883	60	7.88	6.52	391	2.26	0.181	0.98	0.63	0.276
"	#13	751	60	5.73	4.87	292	2.57	0.166	0.79	0.55	0.300

(1) Based on sieve analysis of material through which the canals were built.

(2) Based on sieve analysis of bed material.

(3) Estimated from Lane's surface samples.

(4) Estimated from D_{85} and D_{50} .

APPENDIX C , FIGURES

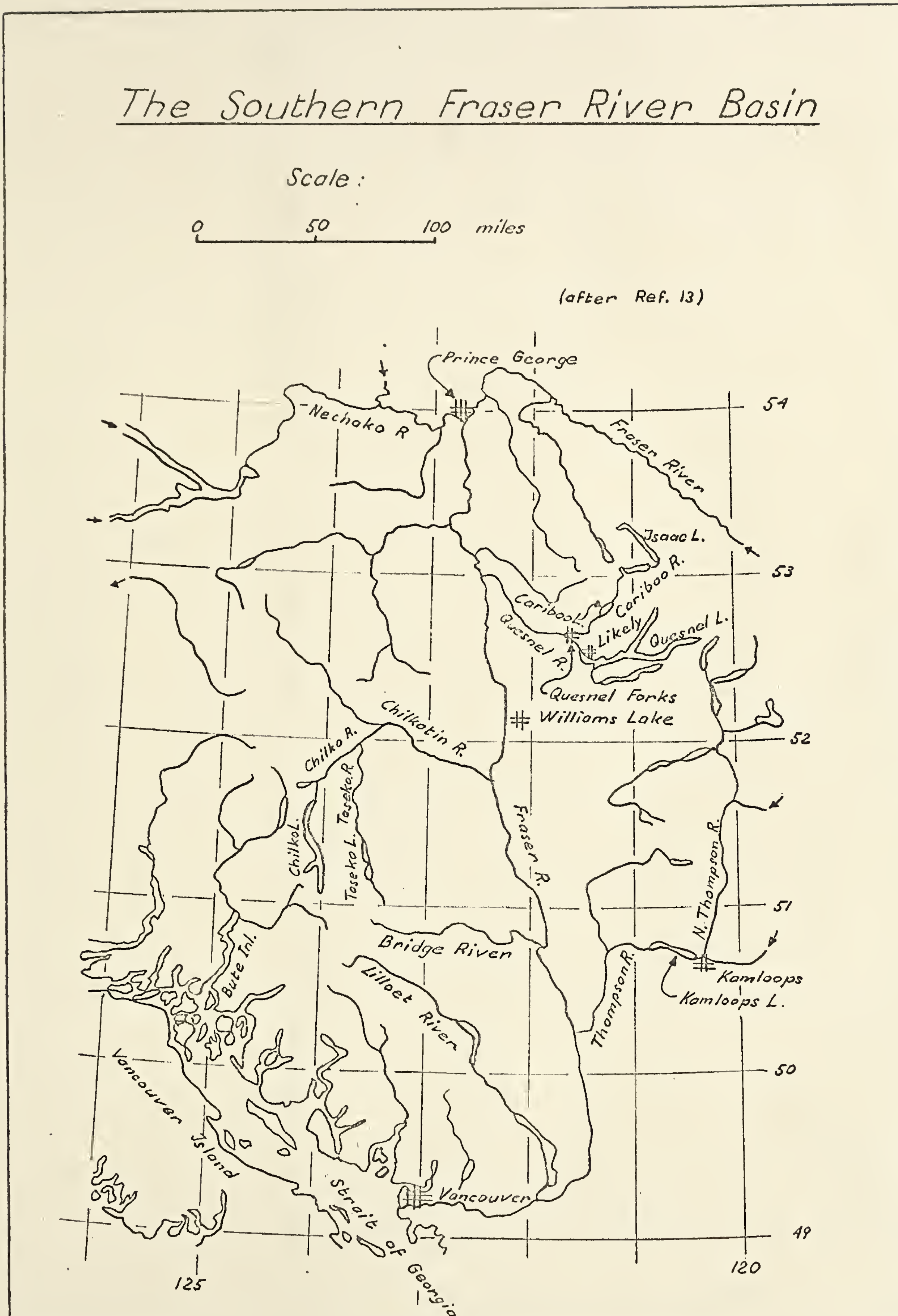
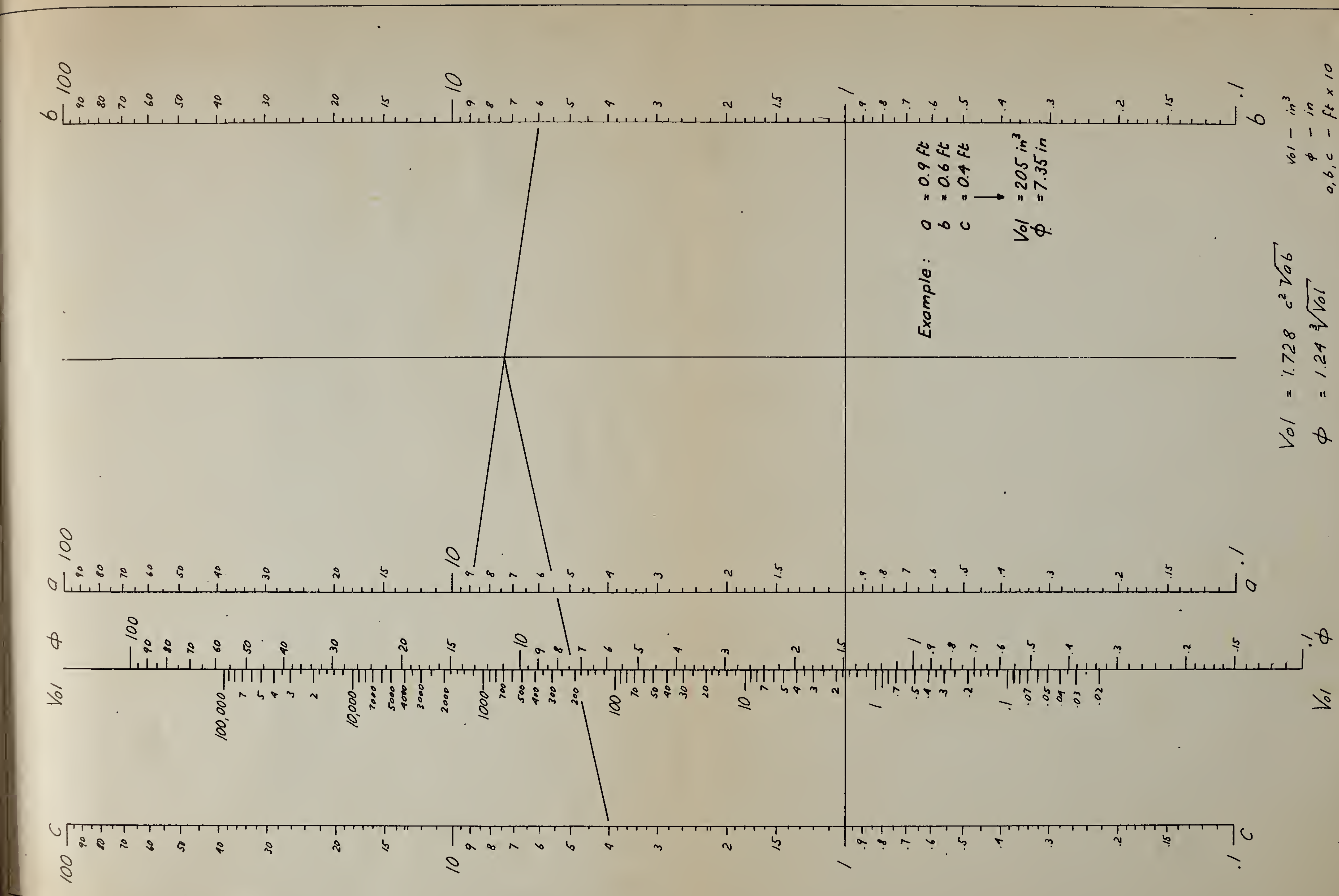


Fig. 1



b
a
φ
Vol
c
Example:
a = 0.9 ft
b = 0.6 ft
c = 0.4 ft
Vol = 205 in³
φ = 7.35 in
Vol = 1.728 c² √ab
φ = 1.24 ∛Vol
Vol – in³
φ – in
a, b, c – ft x 10
Nov 21 62 RK

Fig. 2

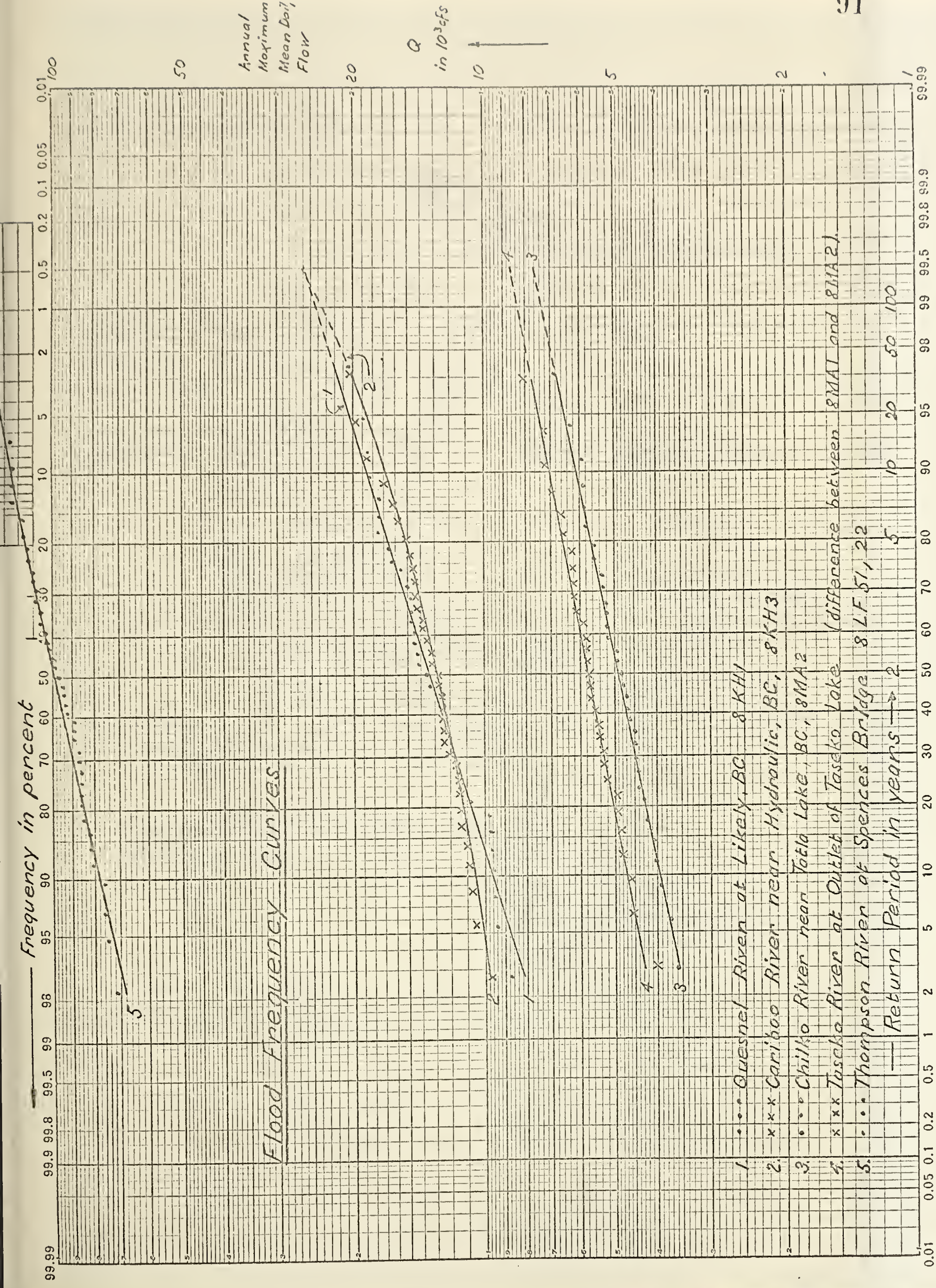

Frequency in percent
Flood Frequency Curves
Annual Maximum Mean Daily Flow
Q in 10^3 cfs
1. Quesnel River at Likely, BC 8KH1
2. Cariboo River near Hydraulic, BC, 8KH3
3. Chilko River near Tatla Lake, BC, 8MA2
4. Taseko River at Outlet of Taseko Lake (difference between 8MA1 and 8MA2)
5. Thompson River at Spences Bridge 8LF51, 22
Return Period in years

Fig. 3

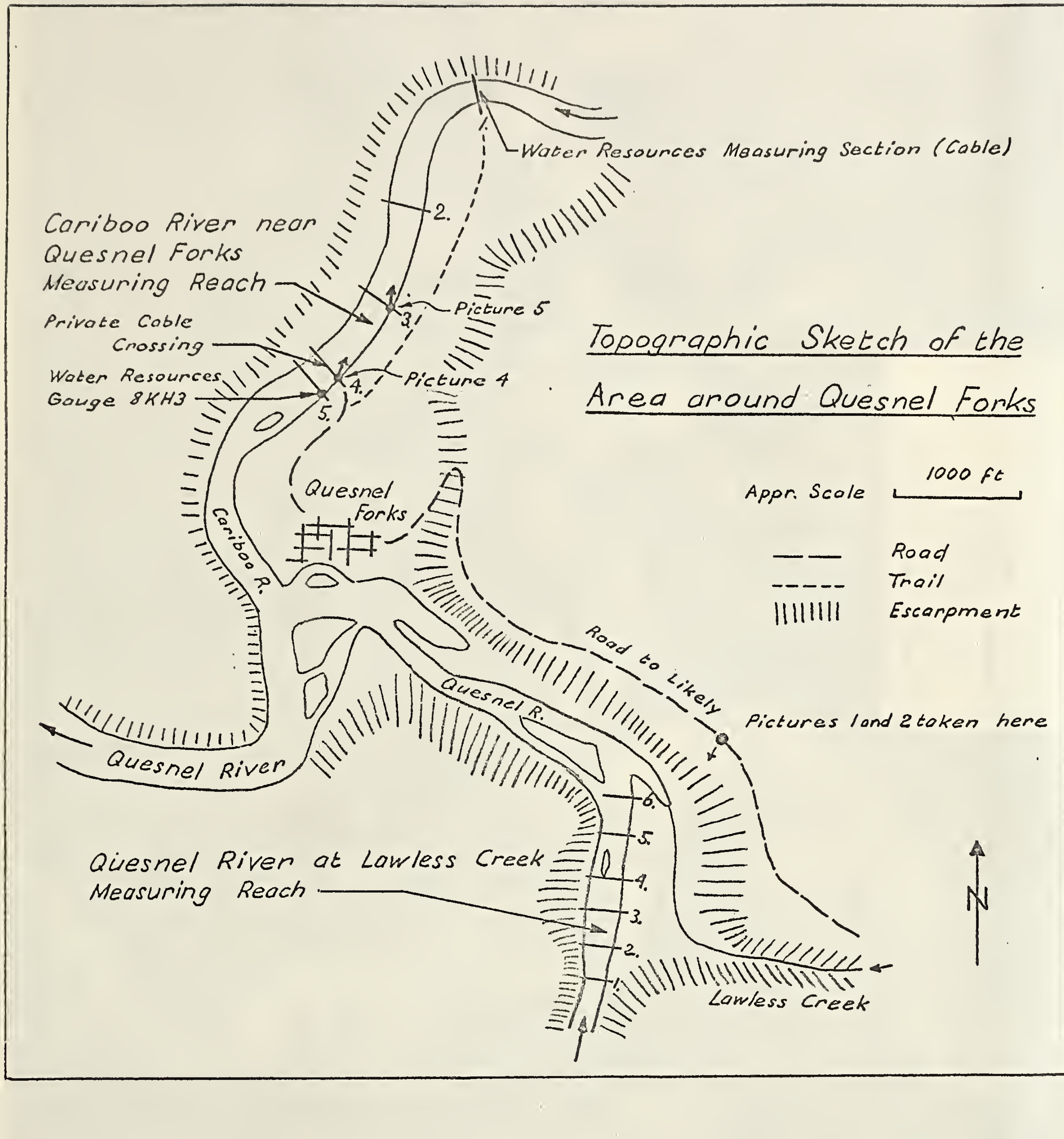

Topographic Sketch of the Area around Quesnel Forks
Water Resources Measuring Section (Cable)
Cariboo River near Quesnel Forks Measuring Reach
Private Cable Crossing
Water Resources Gauge 8KH3
Picture 5
Picture 4
2.
3.
4.
5.
Quesnel Forks
Cariboo R.
Appr. Scale
1000 ft
Road
Trail
Escarpment
Road to Likely
Pictures 1 and 2 taken here
Quesnel R.
Quesnel River
Quesnel River at Lawless Creek Measuring Reach
1.
2.
3.
4.
5.
6.
Lawless Creek
N

Fig. 4

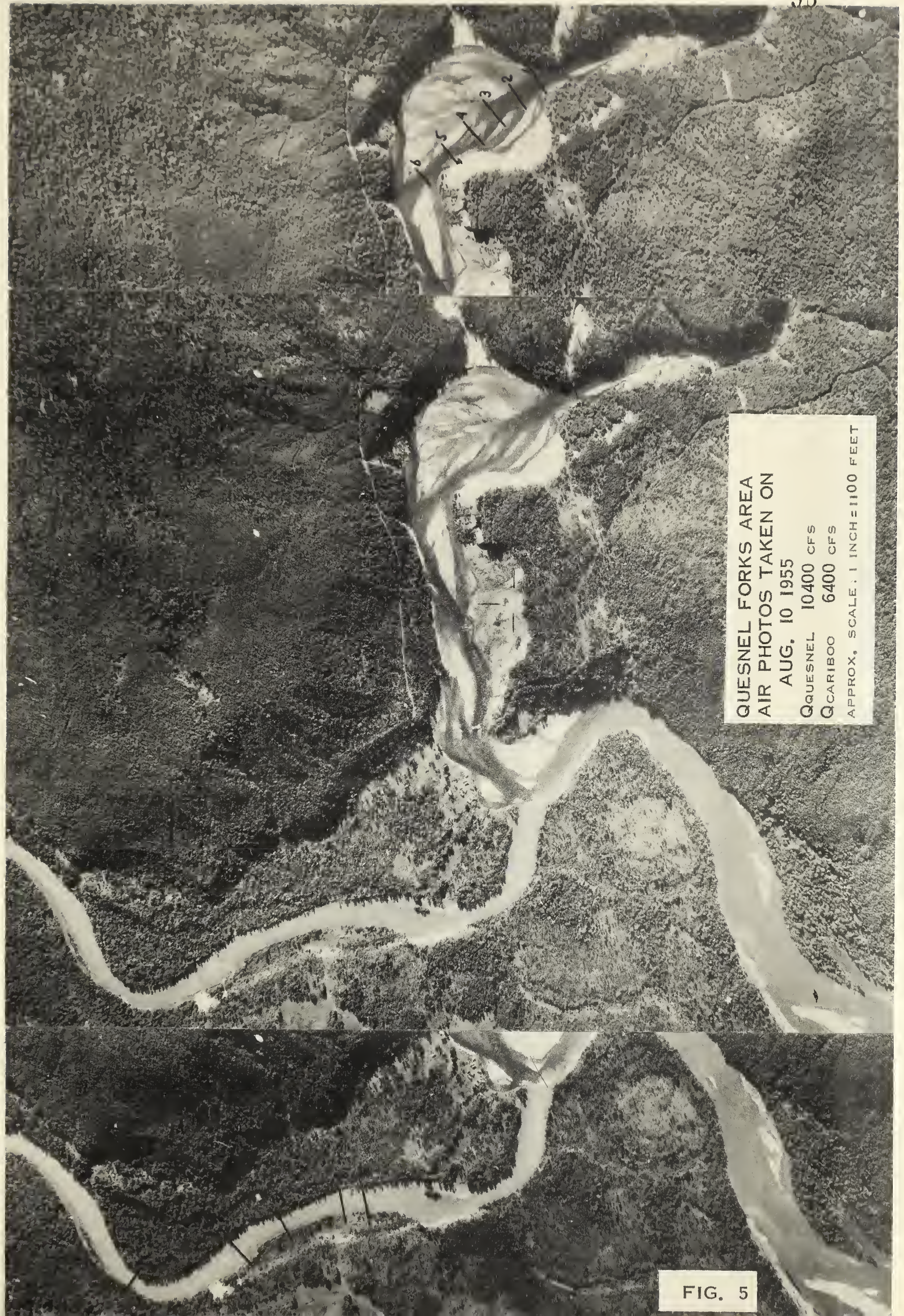

QUESNEL FORKS AREA
AIR PHOTOS TAKEN ON
AUG. 10 1955
$Q_{QUESNEL}$ 10400 CFS
$Q_{CARIBOO}$ 6400 CFS
APPROX. SCALE. 1 INCH = 1100 FEET
1
2
3
4
5
6

FIG. 5

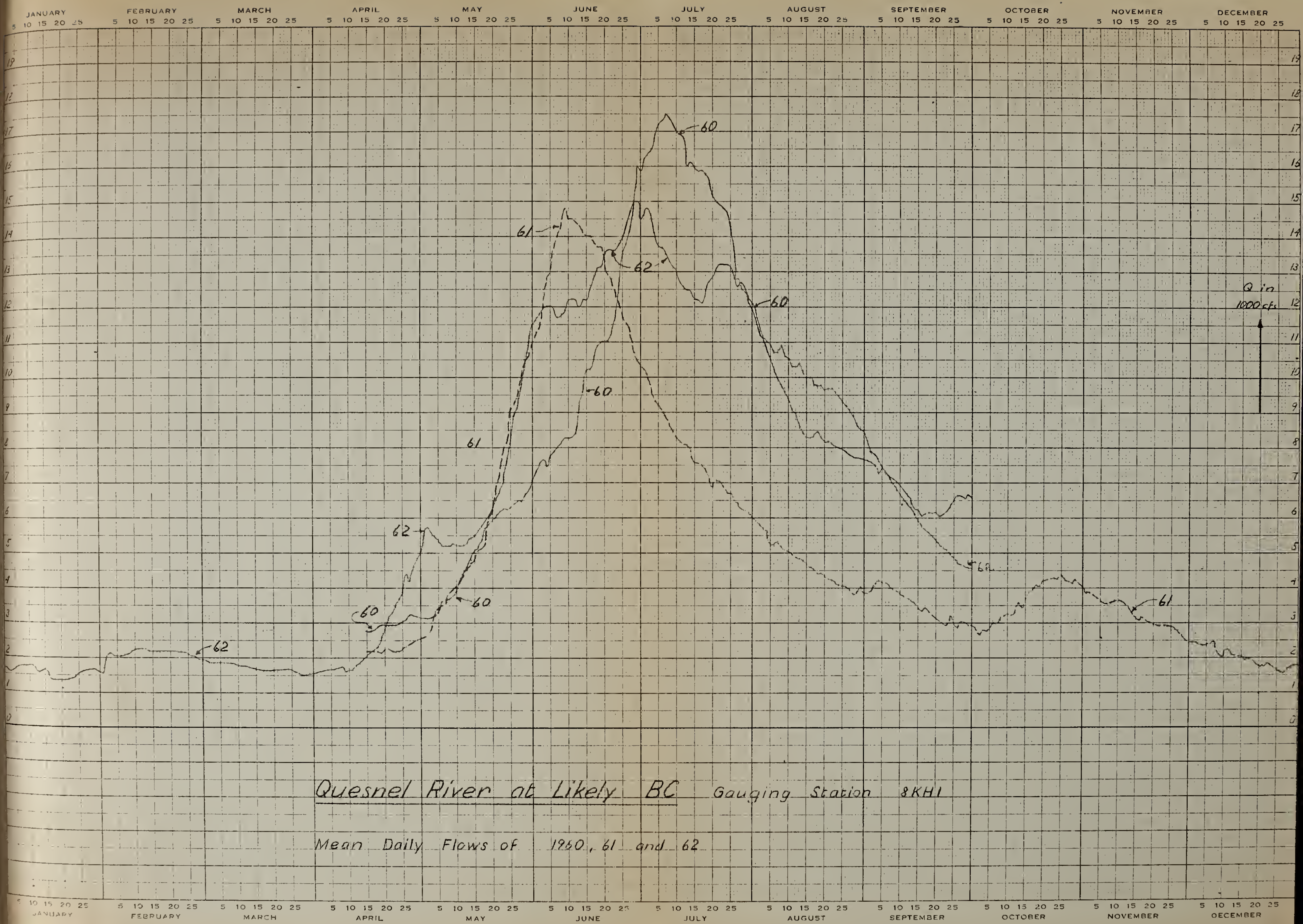

JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
60
61
62
Q in 1000 cfs
Quesnel River at Likely BC Gauging Station 8KH1
Mean Daily Flows of 1960, 61 and 62

Fig. 6

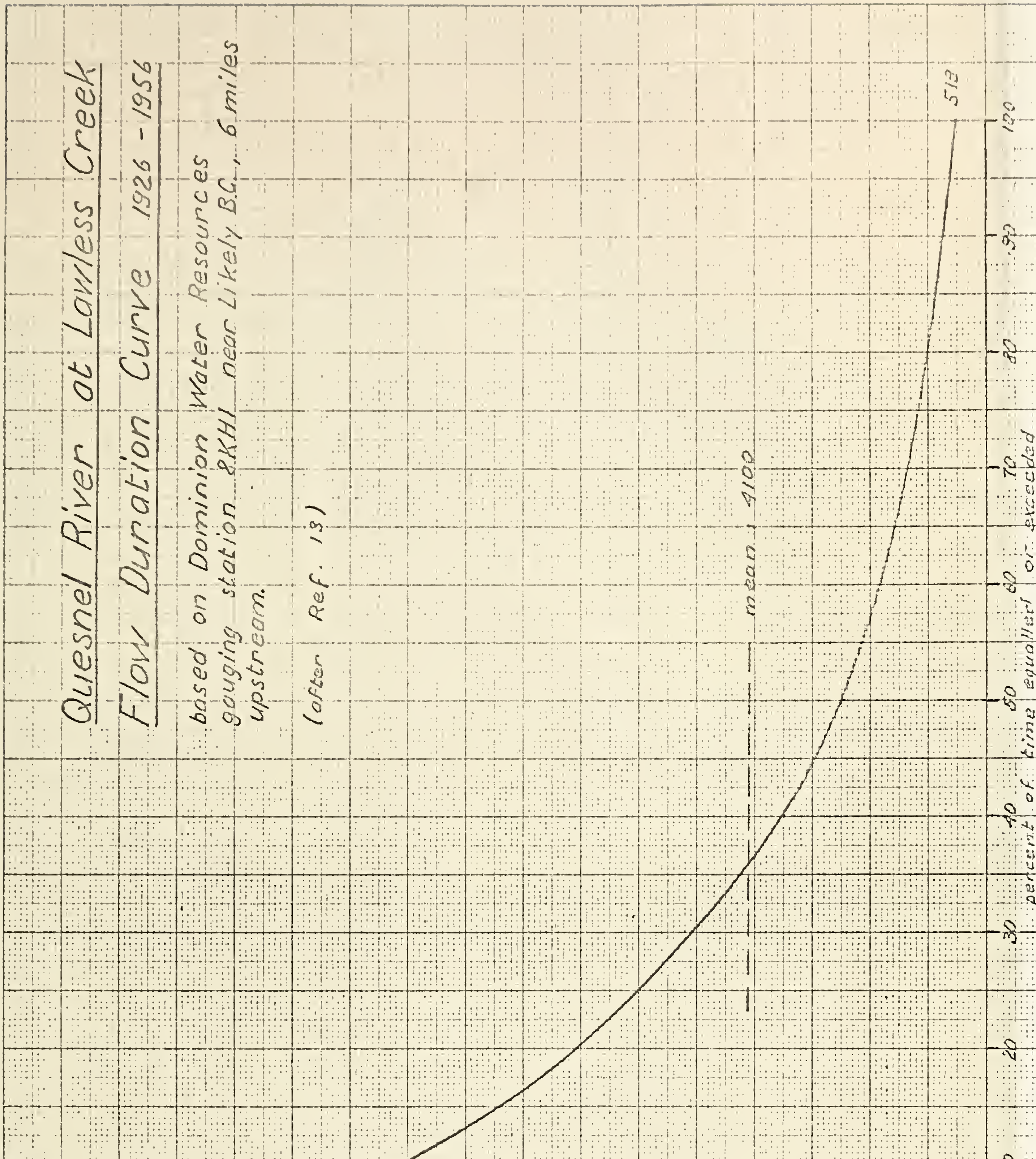
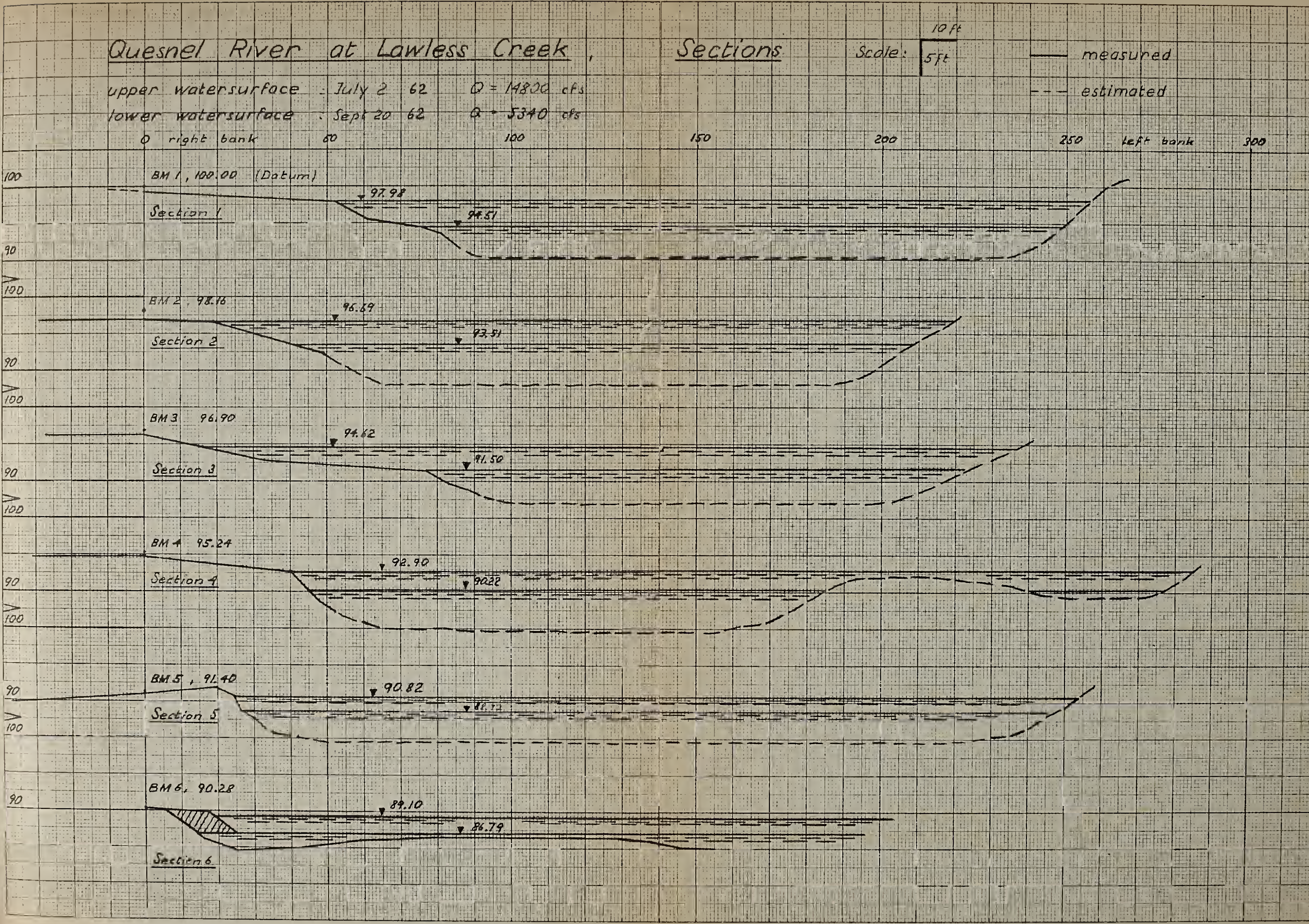

Quesnel River at Lawless Creek
Flow Duration Curve 1926 - 1956
based on Dominion Water Resources gauging station 8KH1 near Likely B.C., 6 miles upstream.
(after Ref. 13)
mean: 4100
16,700
512
0
10
20
30
40
50
60
70
80
90
100
percent of time equalled or exceeded
0
5
10
15
mean monthly discharge in cfs · 10³

Fig. 7



Quesnel River at Lawless Creek, Sections
Scale: 10 ft 5 ft
measured
estimated
upper watersurface : July 2 62 Q = 14800 cfs
lower watersurface : Sept 20 62 Q = 5340 cfs
0 right bank
50
100
150
200
250
left bank
300
BM 1, 100.00 (Datum)
97.98
94.51
Section 1
BM 2, 98.16
96.69
93.51
Section 2
BM 3 96.90
94.62
91.50
Section 3
BM 4 95.24
92.90
90.22
Section 4
BM 5, 91.40
90.82
Section 5
BM 6, 90.28
89.10
86.79
Section 6

Fig. 8

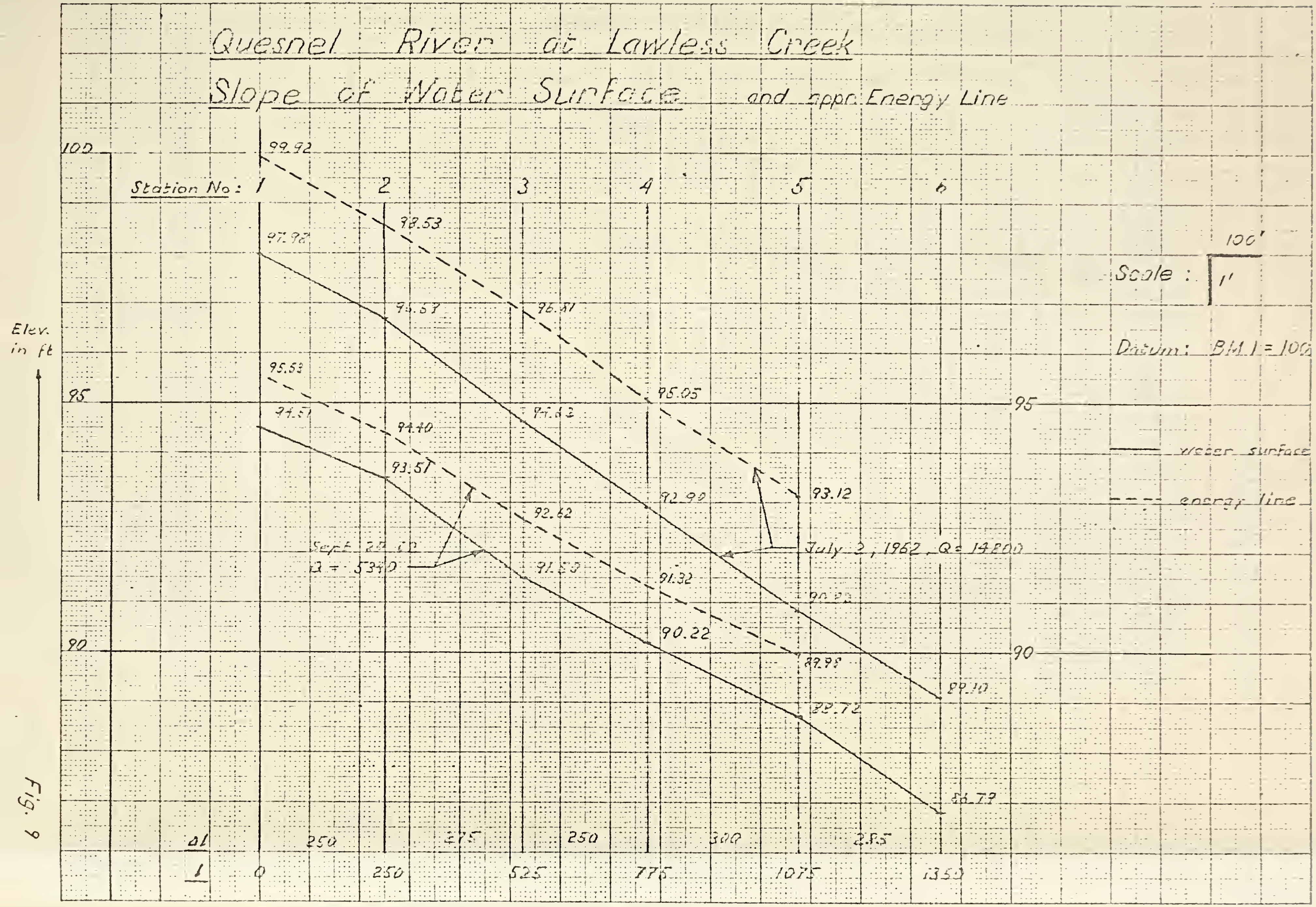

Quesnel River at Lawless Creek
Slope of Water Surface and appr. Energy Line
Station No: 1 2 3 4 5 6
Elev. in ft
100
95
90
99.92
98.53
96.81
95.05
93.12
97.98
95.57
94.63
92.99
89.10
95.53
94.40
92.62
91.32
89.98
94.51
93.51
91.53
90.22
88.72
88.79
Sept. 28. 62
Q = 5340
July 2, 1962, Q = 14800
Scale: 100' / 1'
Datum: BM 1 = 100
water surface
energy line
ΔL 250 275 250 300 285
L 0 250 525 775 1075 1350

Fig. 9

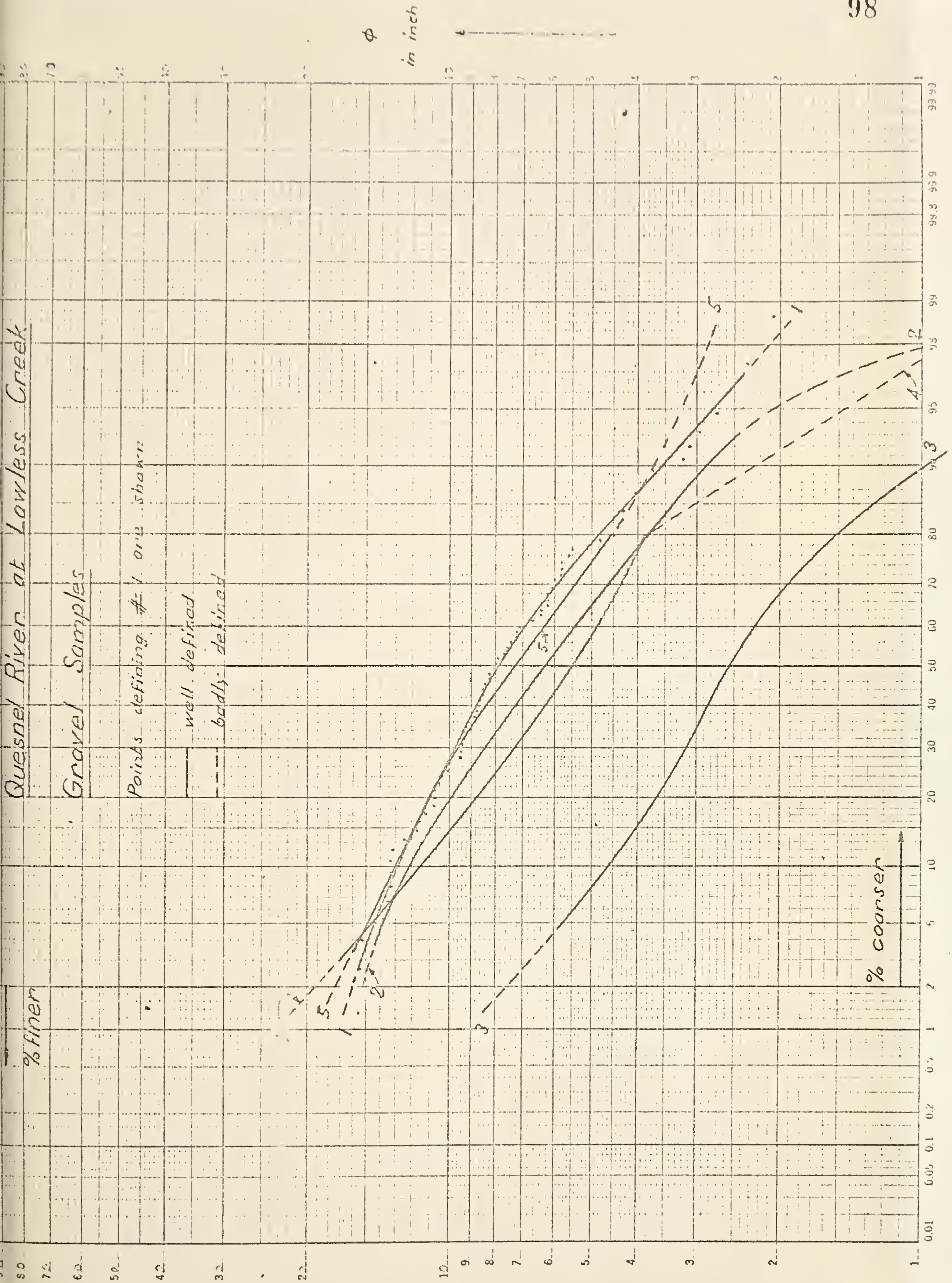
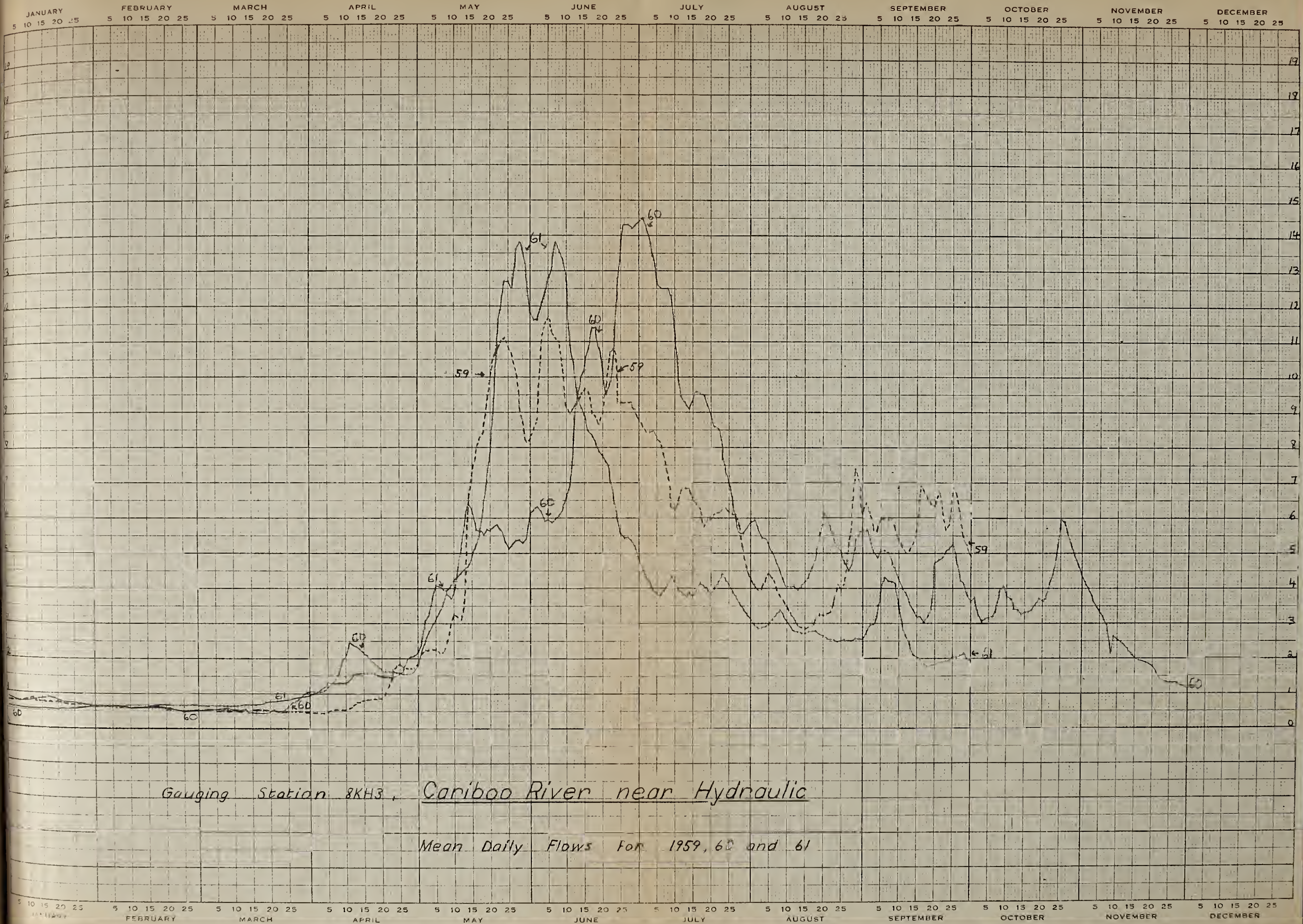

Quesnel River at Lawless Creek
Gravel Samples
Points defining #1 are shown
well defined
badly defined
% finer
% coarser
ϕ in inch

Fig. 10



JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
59
60
61
Gauging Station 8KH3, Cariboo River near Hydraulic
Mean Daily Flows for 1959, 60 and 61

Fig. 11

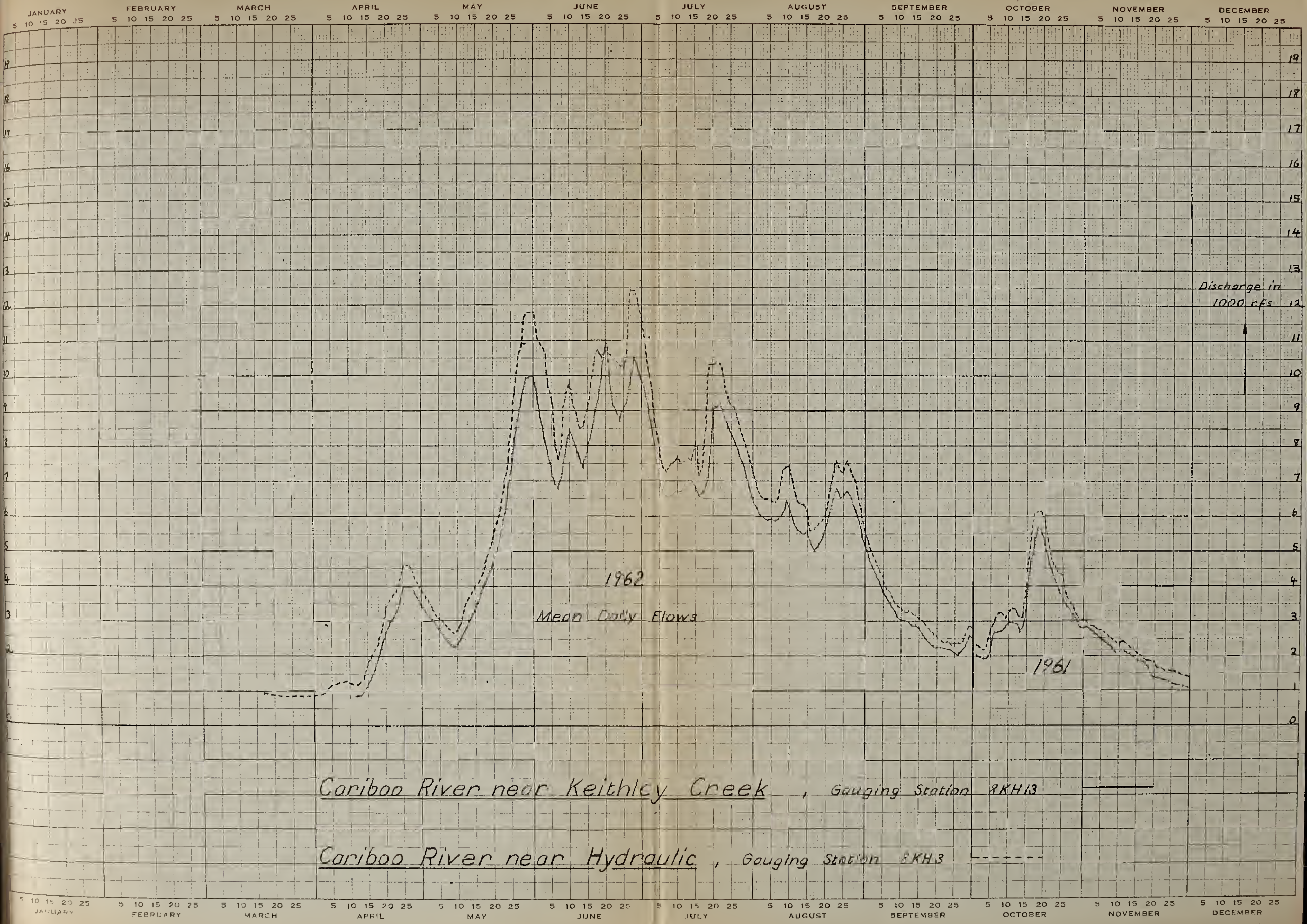

JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
Discharge in 1000 cfs
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
1962
Mean Daily Flows
1961
Cariboo River near Keithley Creek, Gauging Station 8KH13
Cariboo River near Hydraulic, Gauging Station 8KH3

Fig. 12

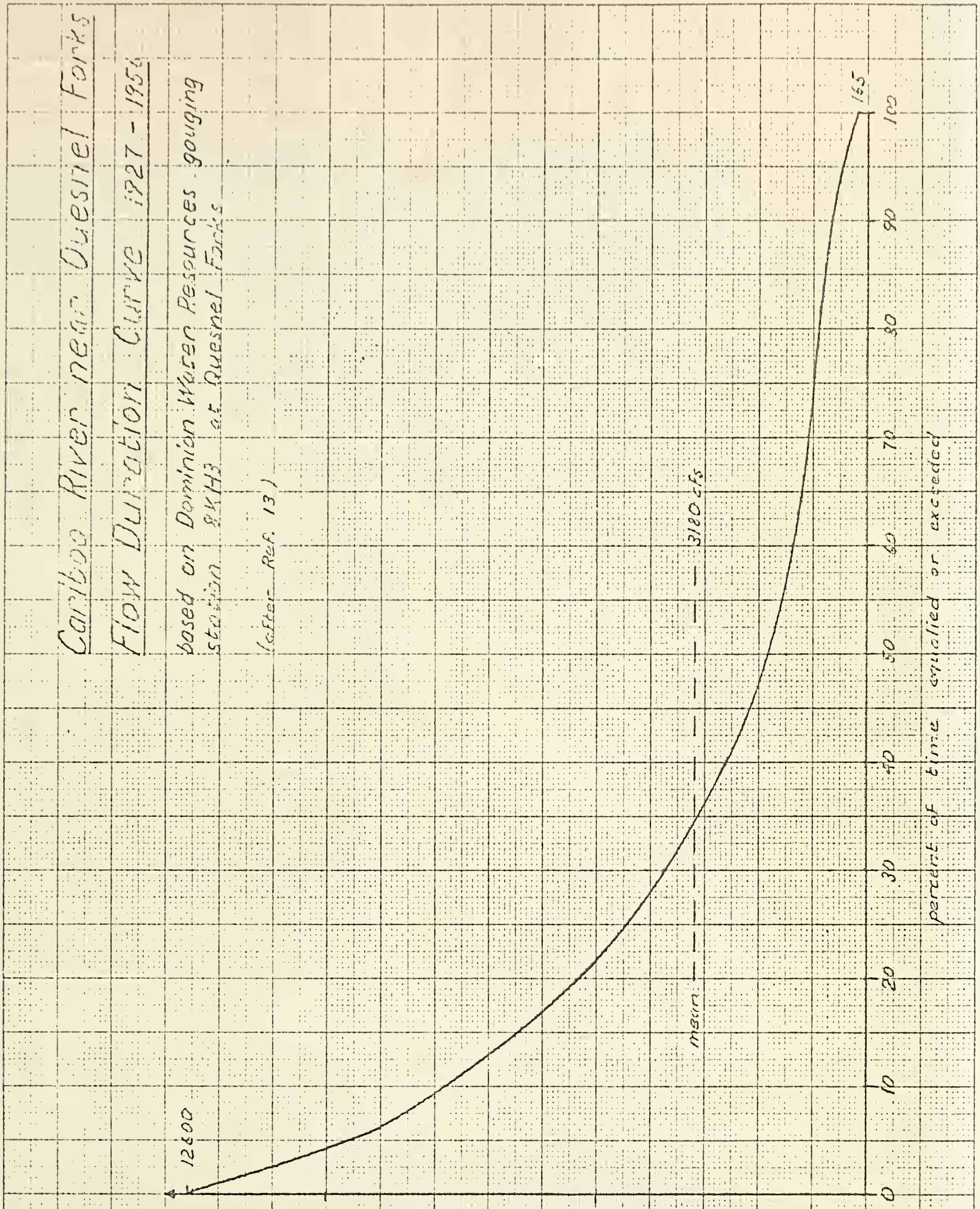
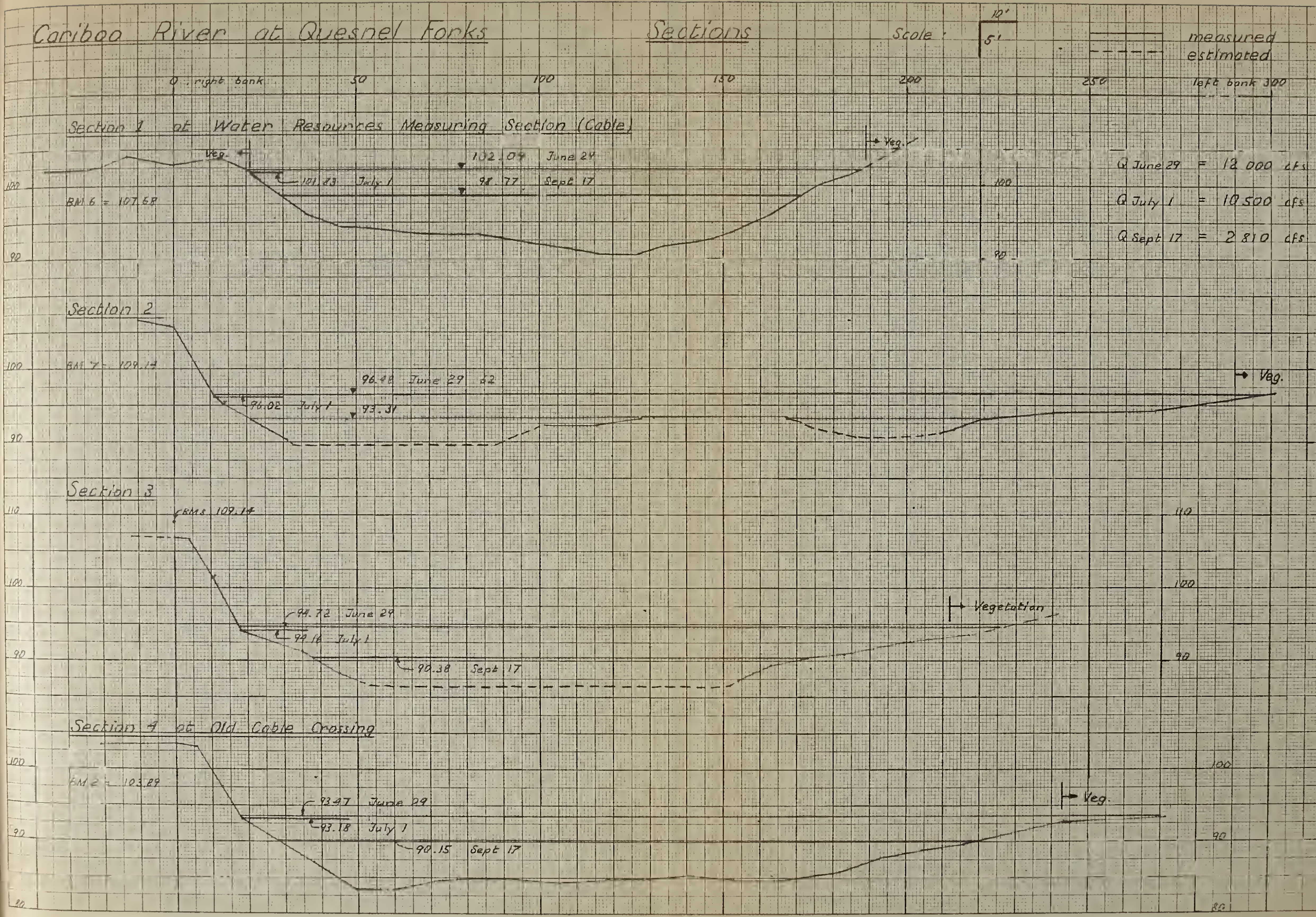

Cariboo River near Quesnel Forks
Flow Duration Curve 1927 - 1956
based on Dominion Water Resources gauging station 8KH3 at Quesnel Forks
(after Ref. 13)
12800
165
3180 c.f.s.
mean
0
10
20
30
40
50
60
70
80
90
100
percent of time equalled or exceeded
10
5
0
mean monthly discharge in c.f.s. 10^3

Fig. 13



Cariboo River at Quesnel Forks
Sections
Scale : 10' 5'
measured
estimated
0 right bank
50
100
150
200
250
left bank 300
Section 1 at Water Resources Measuring Section (Cable)
Veg.
102.04 June 29
101.83 July 1
98.77 Sept 17
BM 6 = 107.68
Q June 29 = 12 000 cfs
Q July 1 = 10 500 cfs
Q Sept 17 = 2 810 cfs
Section 2
BM 7 = 109.13
96.48 June 29
96.02 July 1
93.31
Veg.
Section 3
BM 8 109.14
94.72 June 29
94.16 July 1
90.38 Sept 17
Vegetation
Section 4 at Old Cable Crossing
BM 2 = 103.29
93.47 June 29
93.18 July 1
90.15 Sept 17
Veg.

Fig. 14

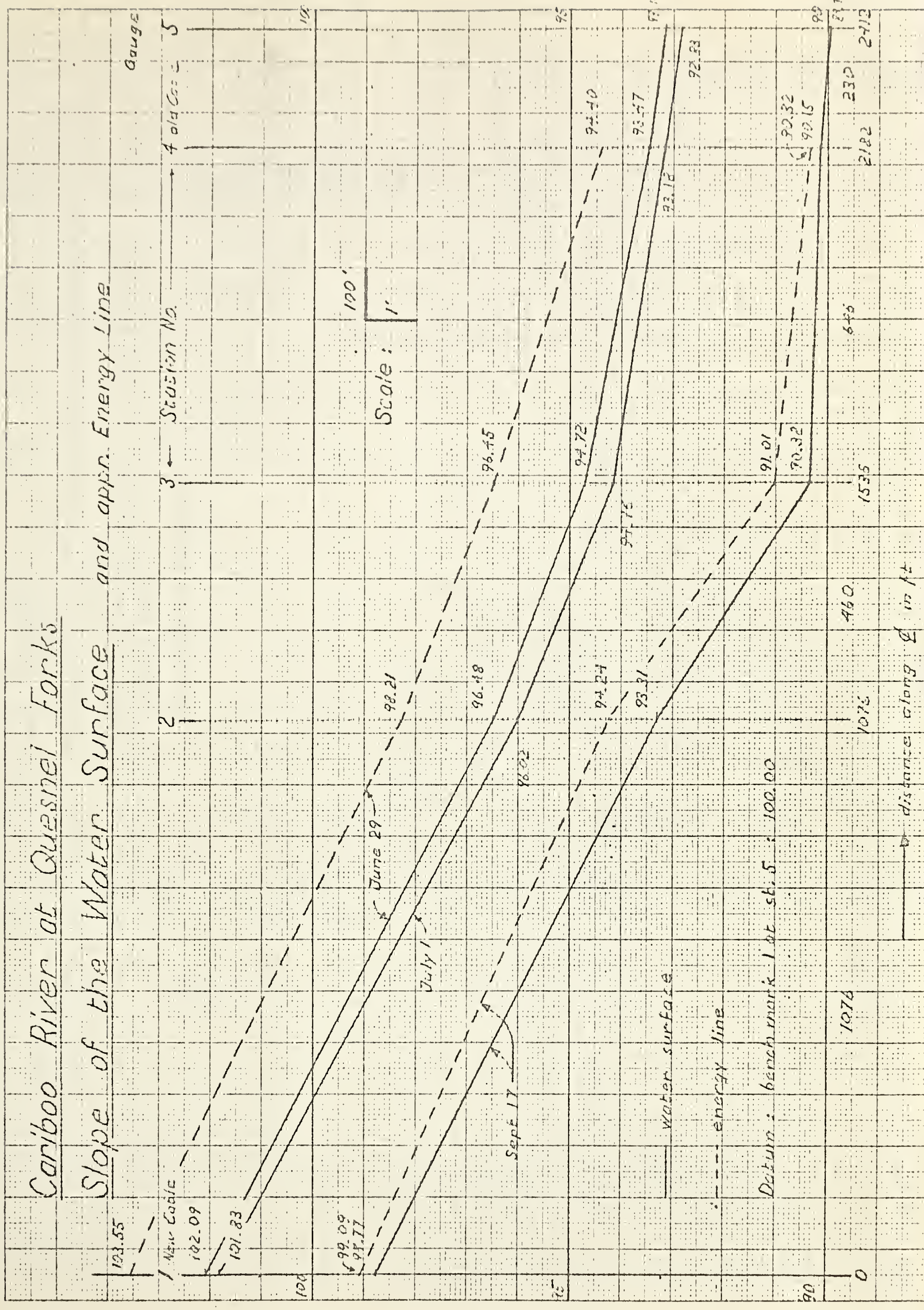

Cariboo River at Quesnel Forks
Slope of the Water Surface and appr. Energy Line
Station No.
Gauge
New Gauge
Scale : 100' 1'
June 29
July 1
Sept 17
water surface
energy line
Datum : benchmark 1 at st. 5 : 100.00
distance along ℄ in ft
Elev. in ft

Fig. 15

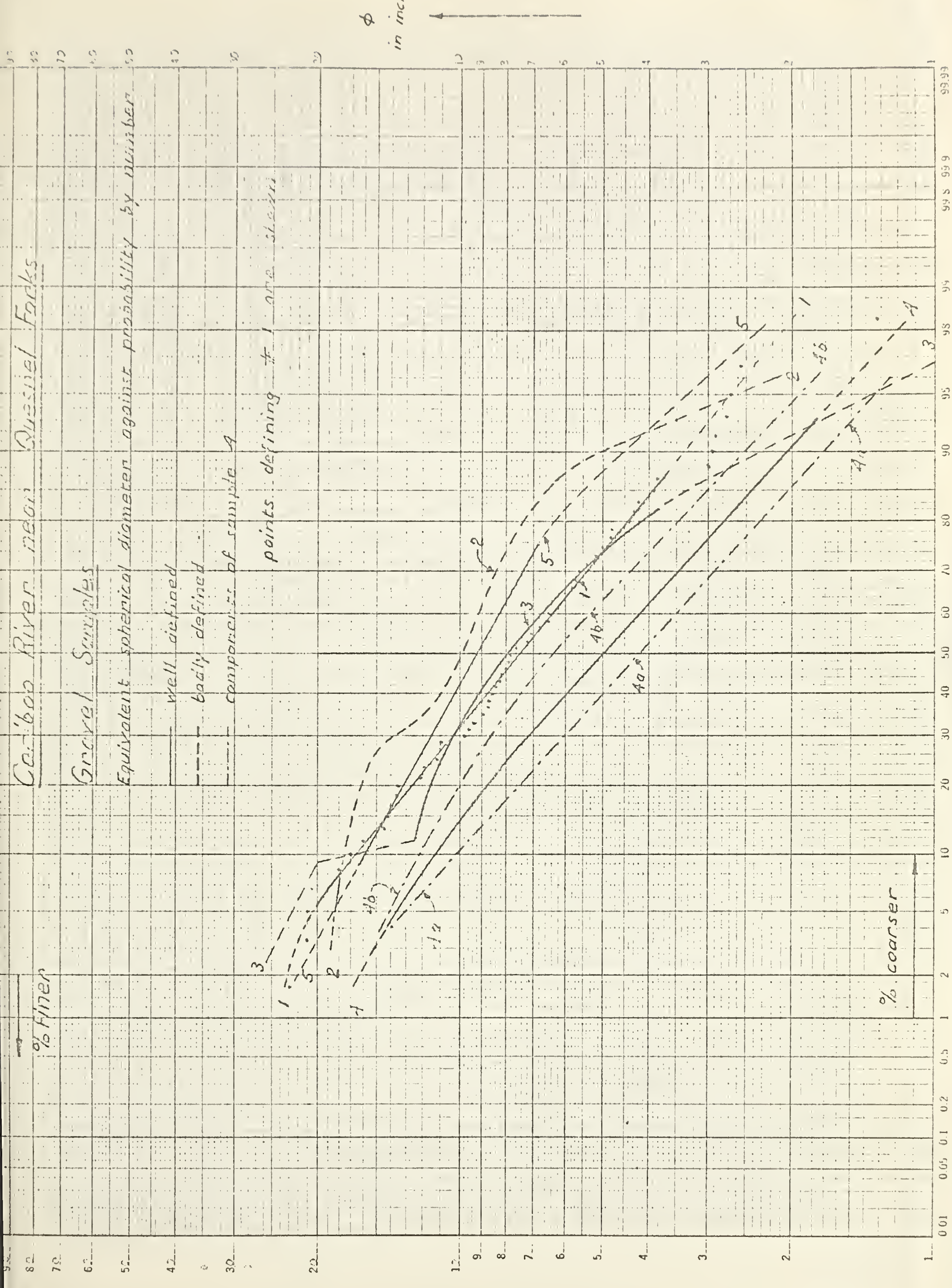

Cariboo River near Quesnel Forks
Gravel Samples
Equivalent spherical diameter against probability by number
well defined
badly defined
components of sample 4
points defining # 1 are shown
% finer
% coarser
ϕ in inch
0.01
0.05
0.1
0.2
0.5
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99
99.8
99.9
99.99
1
2
3
4
5
6
7
8
9
10
20
30
1
2
3
4
5
4a
4b

Fig. 16

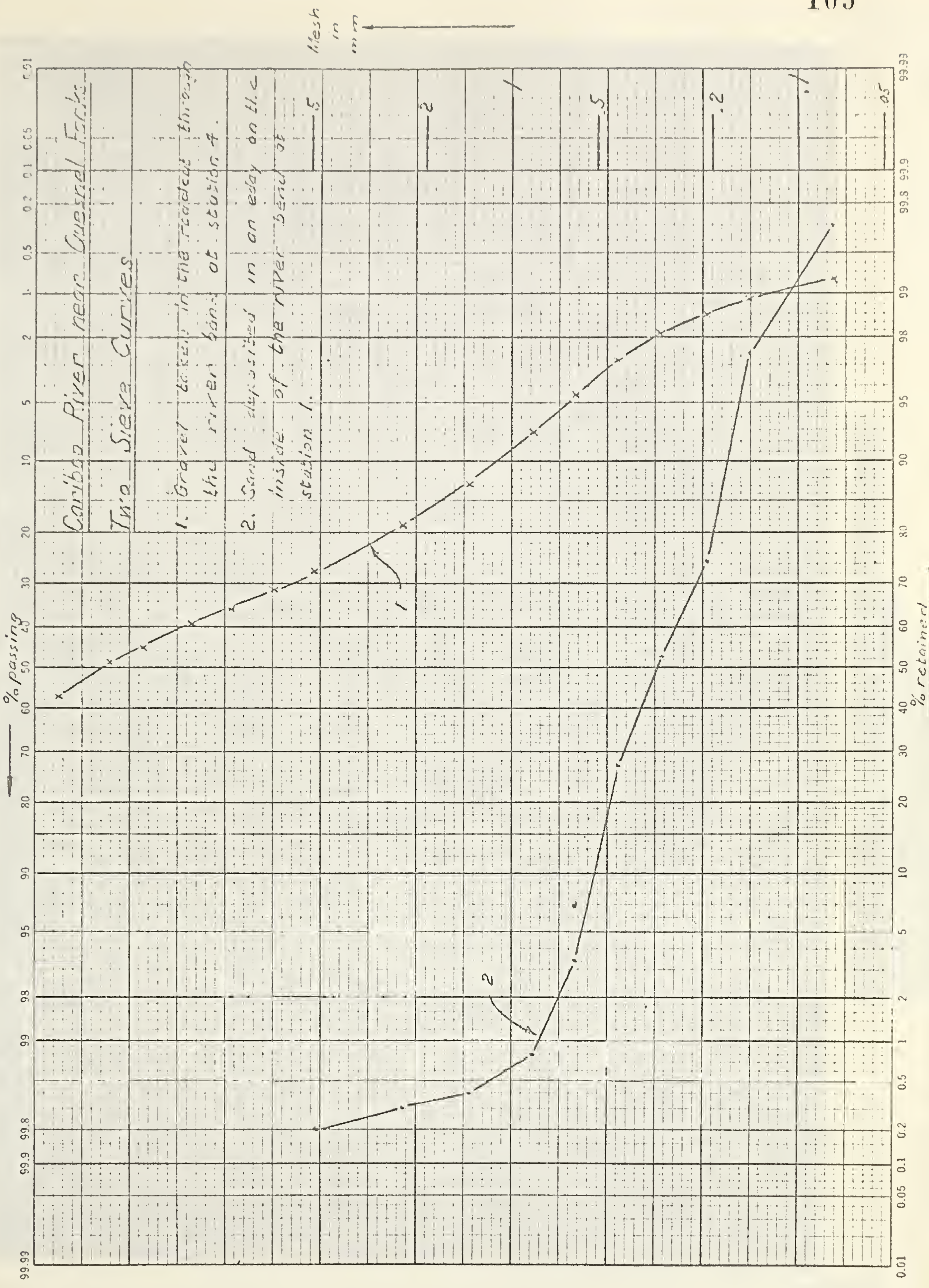
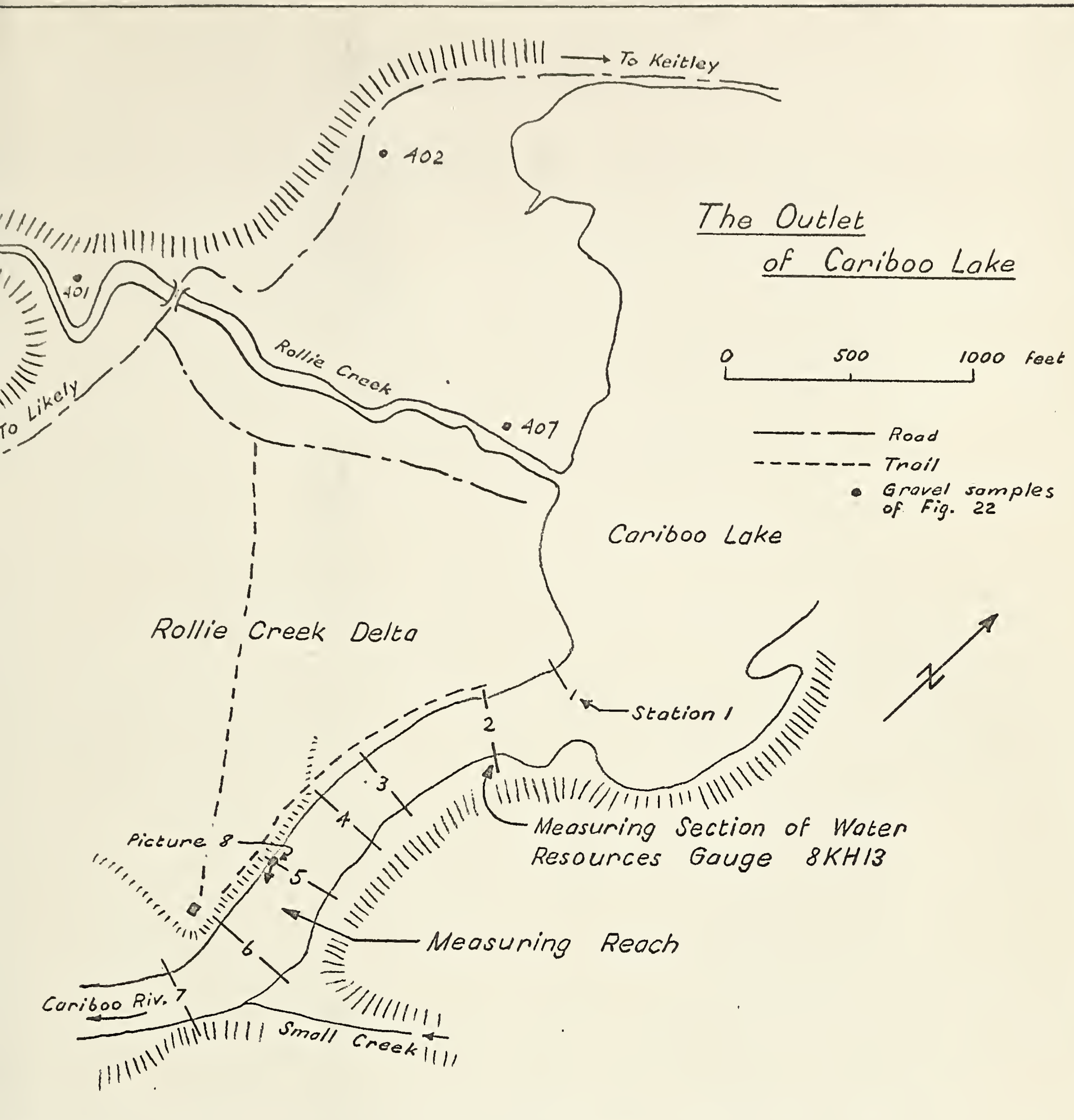

Cariboo River near Quesnel Forks
Two Sieve Curves
1. Gravel taken in the middle through the river bank at station 4.
2. Sand deposited in an eddy on the inside of the river bend at station 1.
Mesh in mm
% passing
% retained
1
2

Fig. 17

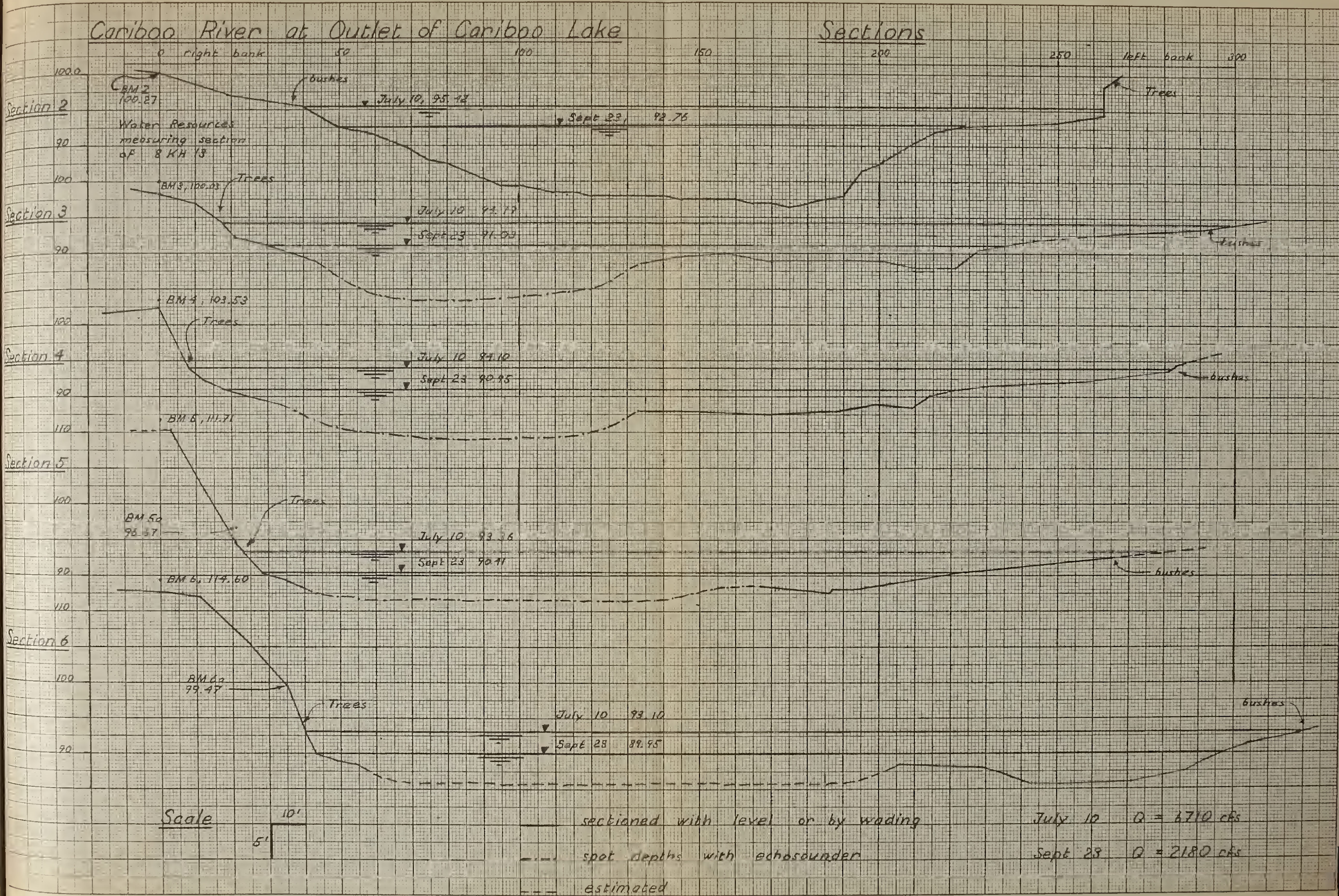
CARIBOO RIVER AT THE OUTLET OF CARIBOO LAKE
AIR PHOTOS TAKEN ON SEPT. 11 1957
$Q_{CARIBOO}$ 2940 CFS
APPROX. SCALE : 1 INCH = 1200 FEET

FIG. 18



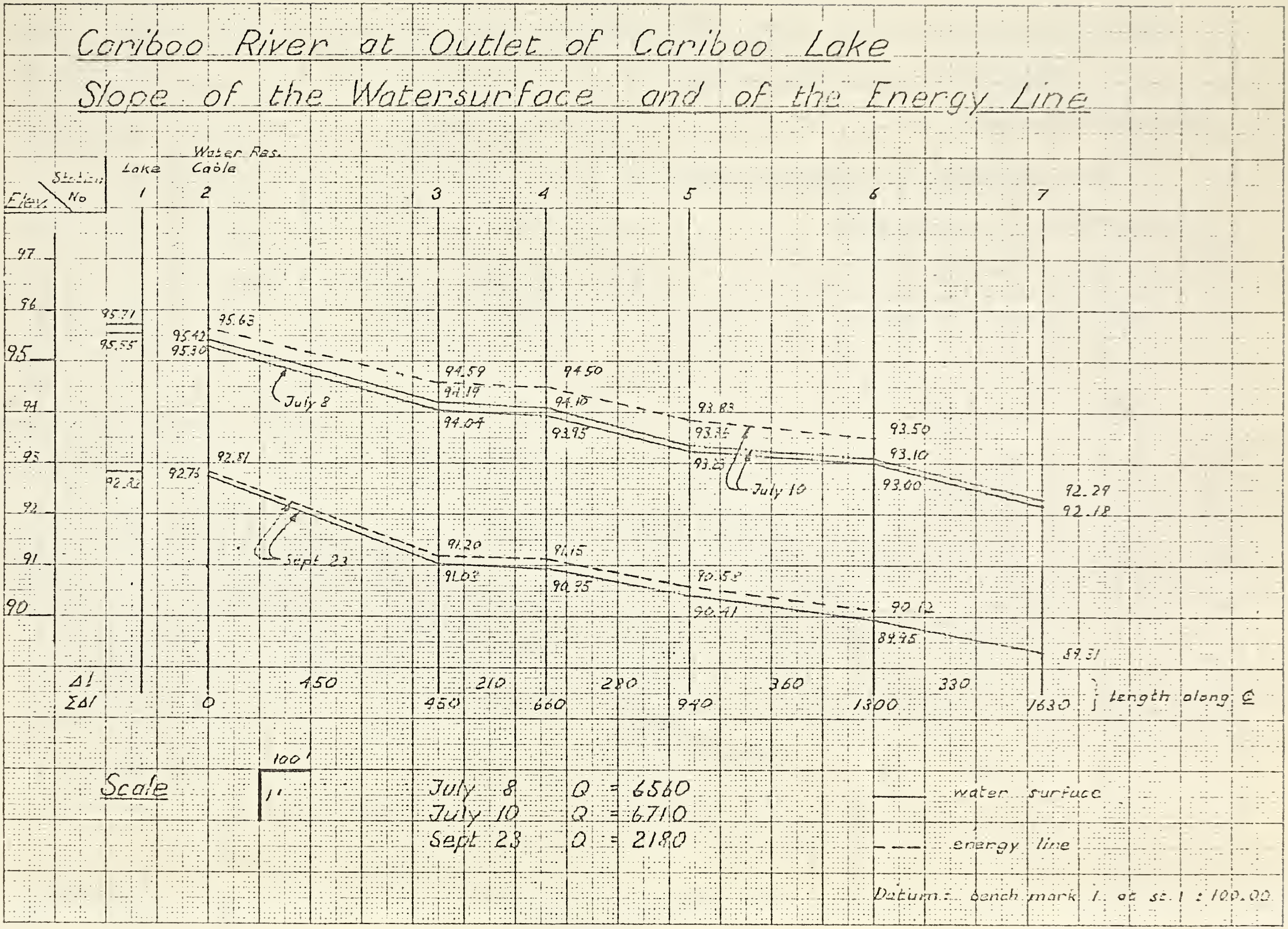
To Keitley
402
The Outlet of Cariboo Lake
401
Rollie Creek
To Likely
0
500
1000 feet
407
Road
Trail
Gravel samples of Fig. 22
Cariboo Lake
Rollie Creek Delta
Station 1
2
3
4
Measuring Section of Water Resources Gauge 8KH13
Picture 8
5
Measuring Reach
6
Cariboo Riv. 7
Small Creek

Fig. 19



Cariboo River at Outlet of Cariboo Lake
Sections
0 right bank
50
100
150
200
250
left bank
300
Section 2
100.0
BM 2
100.27
bushes
July 10, 95.42
Sept 23, 92.76
Trees
Water Resources
measuring section
of 8 KH 13
90
100
Section 3
BM 3, 100.03
Trees
July 10 94.19
Sept 23 91.03
bushes
90
BM 4, 103.53
100
Trees
Section 4
July 10 94.10
Sept 23 90.95
bushes
90
BM 5, 111.71
110
Section 5
100
Trees
BM 5a
95.57
July 10 93.36
Sept 23 90.41
bushes
90
BM 6, 114.60
110
Section 6
100
BM 6a
99.47
Trees
July 10 93.10
Sept 23 89.95
bushes
90
Scale
10'
5'
sectioned with level or by wading
spot depths with echosounder
estimated
July 10 Q = 6710 cfs
Sept 23 Q = 2180 cfs

Fig. 20



Elev. in ft

Fig. 21

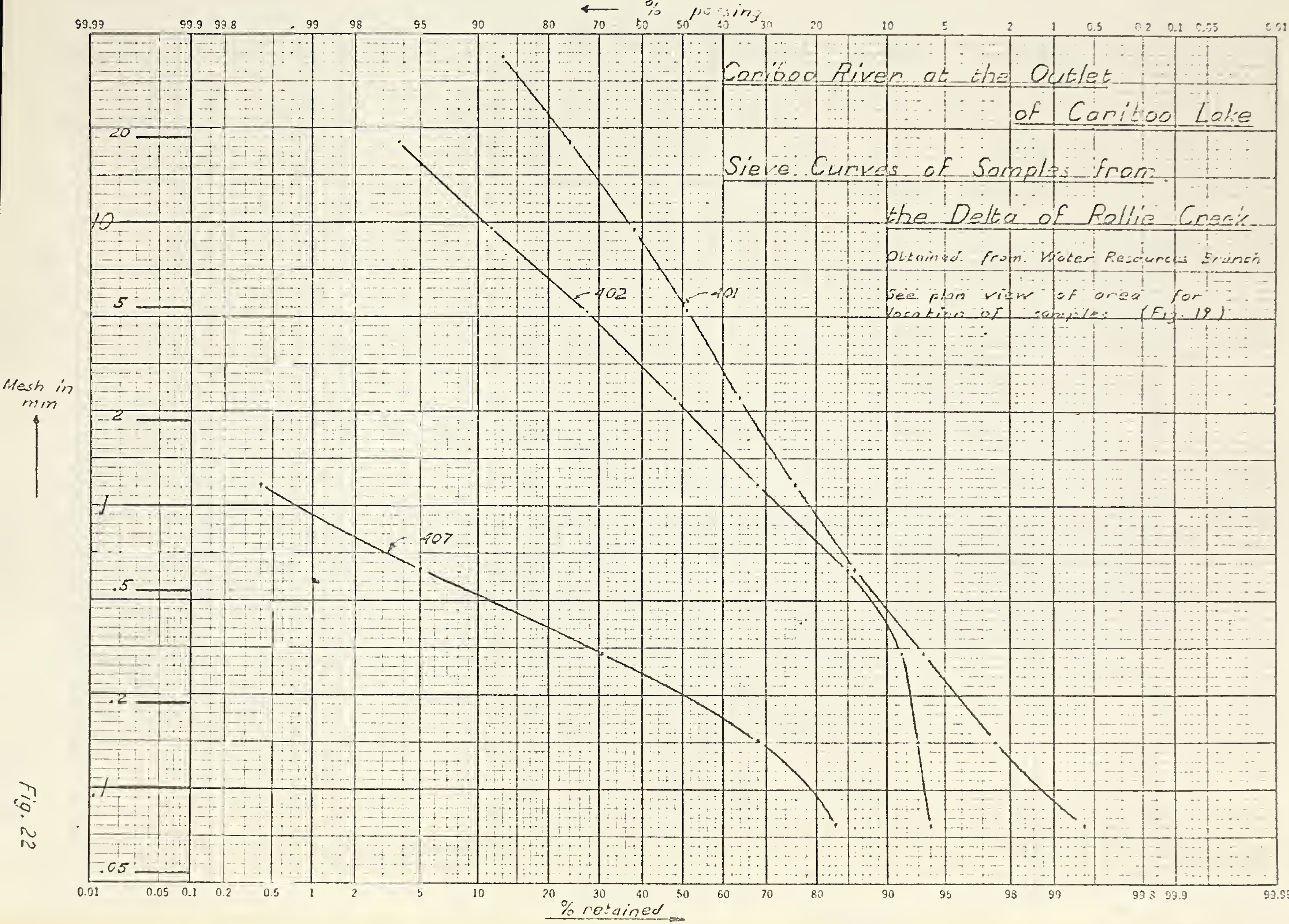

% passing
Cariboo River at the Outlet
of Cariboo Lake
Sieve Curves of Samples from
the Delta of Rollie Creek
Obtained from Water Resources Branch
See plan view of area for location of samples (Fig. 19)
401
402
407
Mesh in mm
% retained

Fig. 22

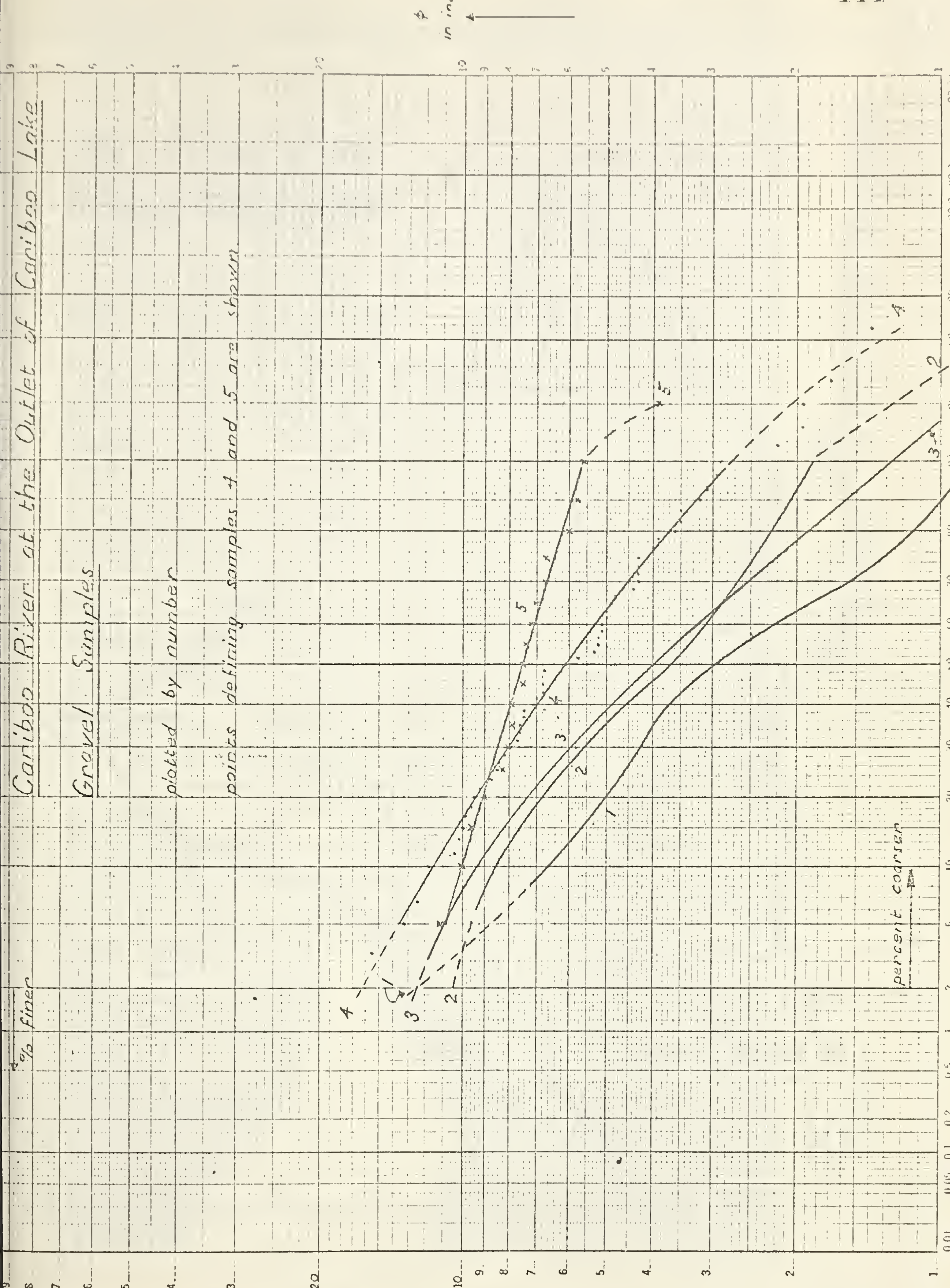

Cariboo River at the Outlet of Cariboo Lake
Gravel Samples
plotted by number
points defining samples 4 and 5 are shown
% finer
percent coarser
in inch

Fig. 23

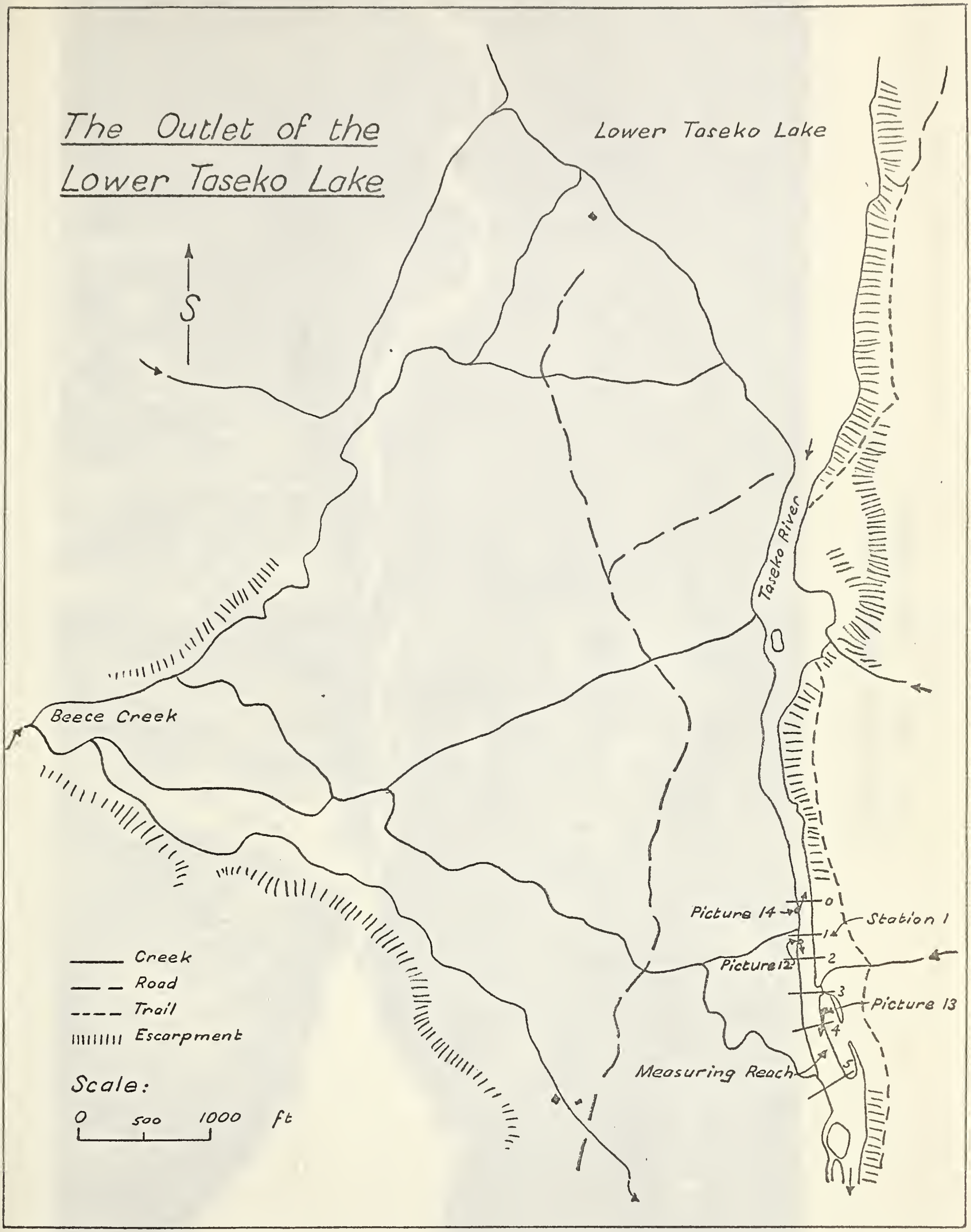
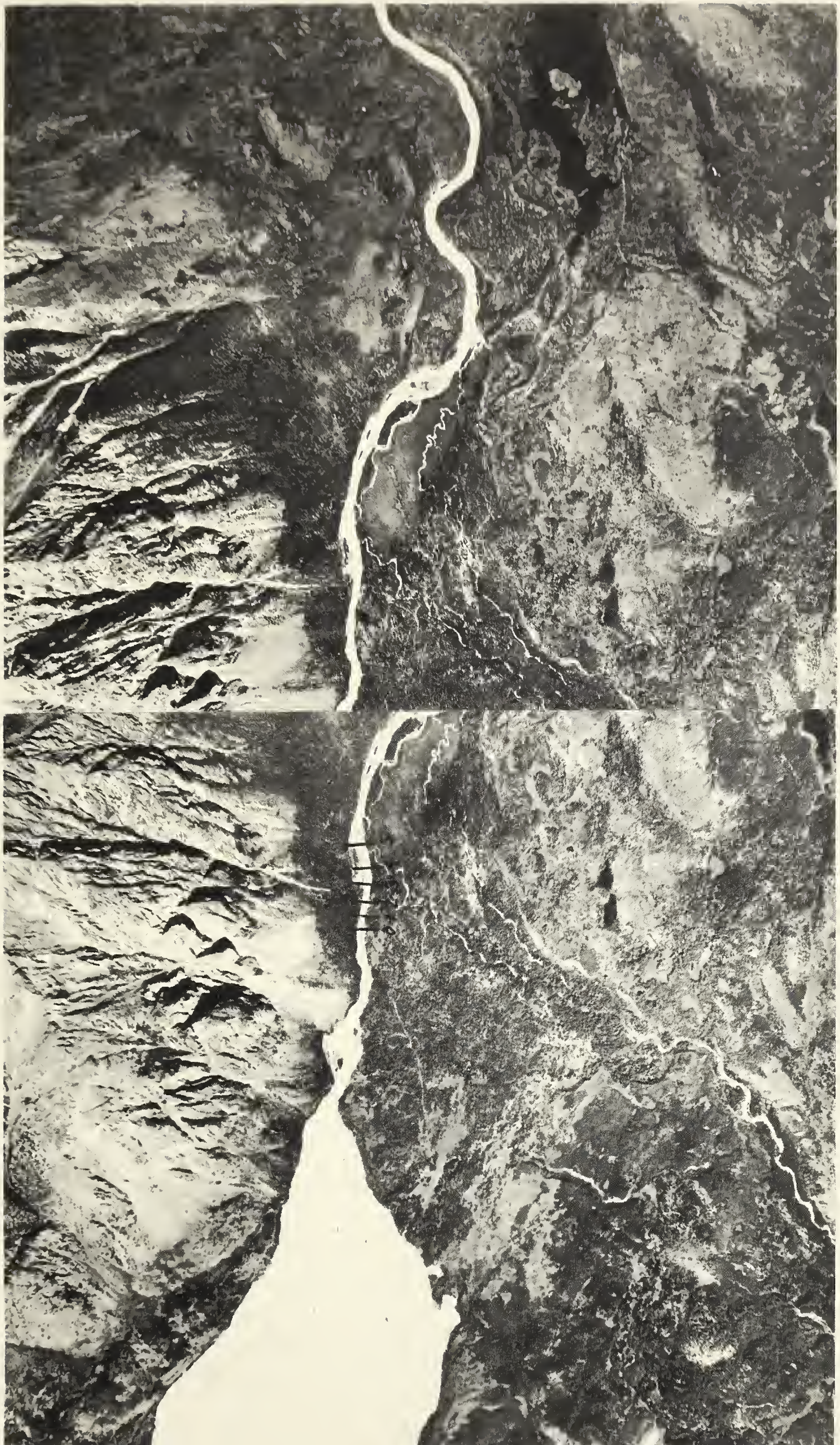

The Outlet of the Lower Taseko Lake
Lower Taseko Lake
S
Taseko River
Beece Creek
Picture 14
Station 1
Picture 12
Picture 13
Measuring Reach
0
1
2
3
4
5
Creek
Road
Trail
Escarpment
Scale:
0 500 1000 ft

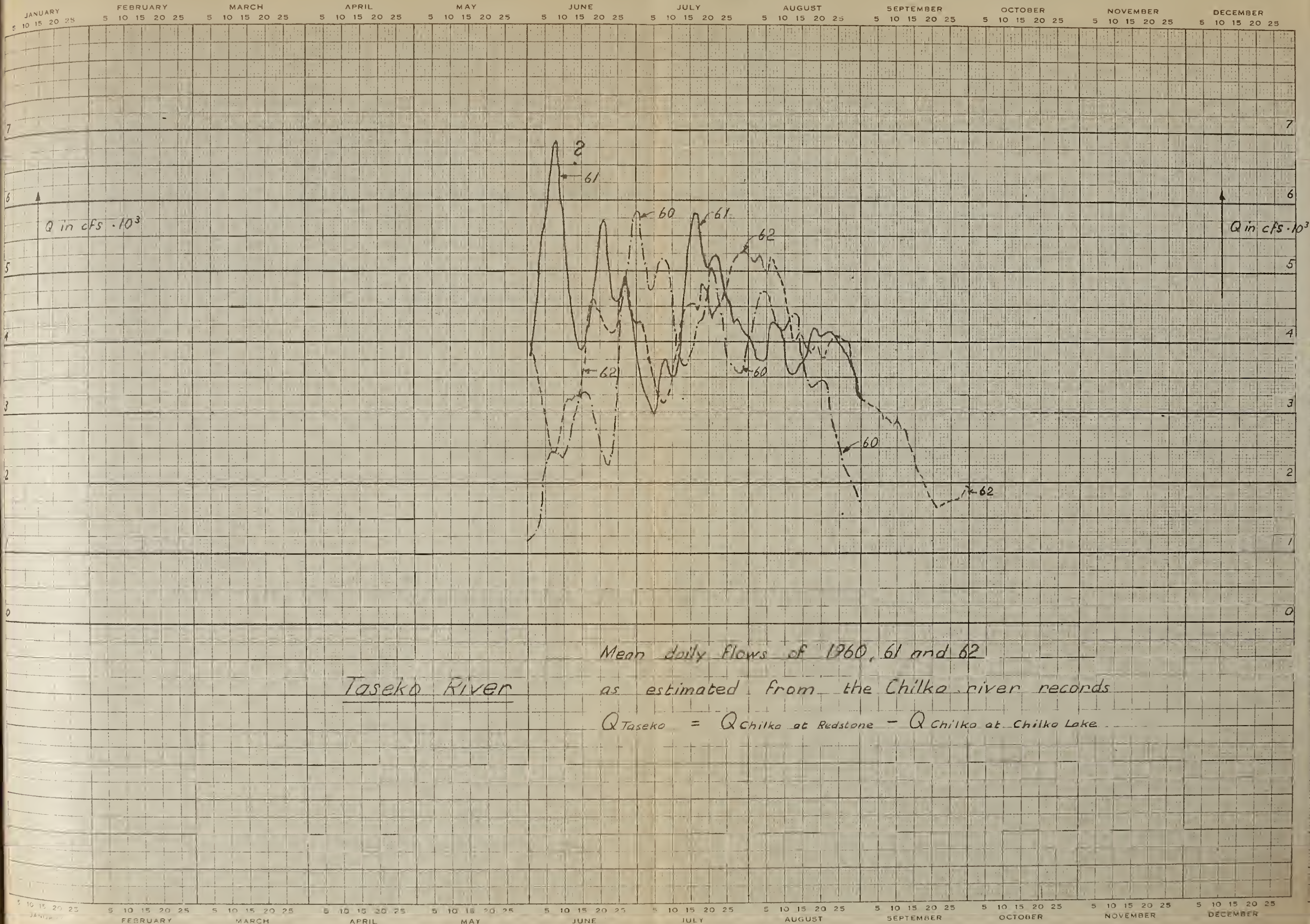
Fig. 24



TASEKO RIVER BELOW TASEKO LAKE

APPROX. SCALE 1 INCH 2700 FEET

Fig. 25



JANUARY
FEBRUARY
MARCH
APRIL
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
5 10 15 20 25
Q in cfs · 10^3
0
1
2
3
4
5
6
7
60
61
62
Taseko River
Mean daily flows of 1960, 61 and 62
as estimated from the Chilko river records
$Q_{Taseko} = Q_{Chilko\ at\ Redstone} - Q_{Chilko\ at\ Chilko\ Lake}$

Fig. 26

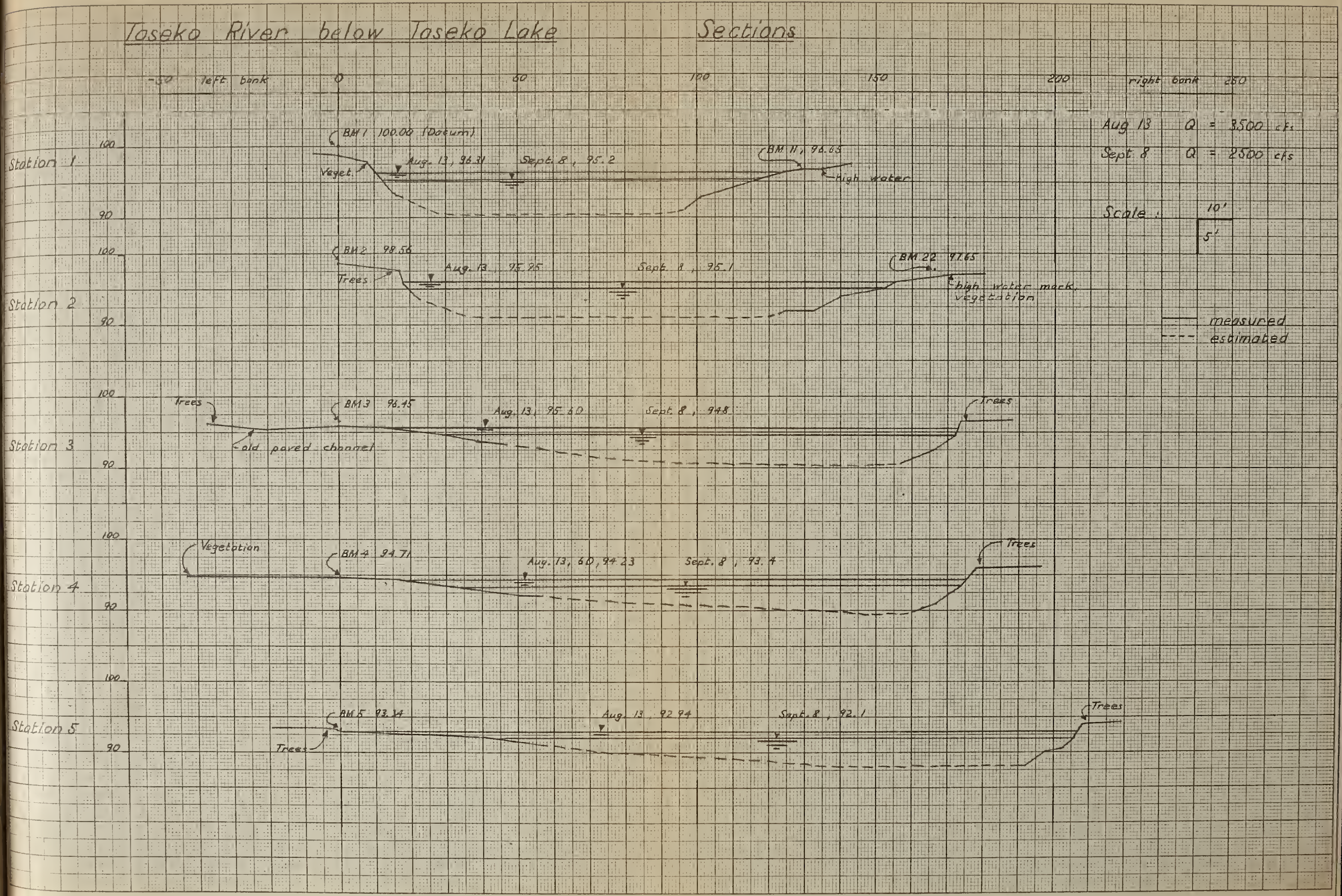
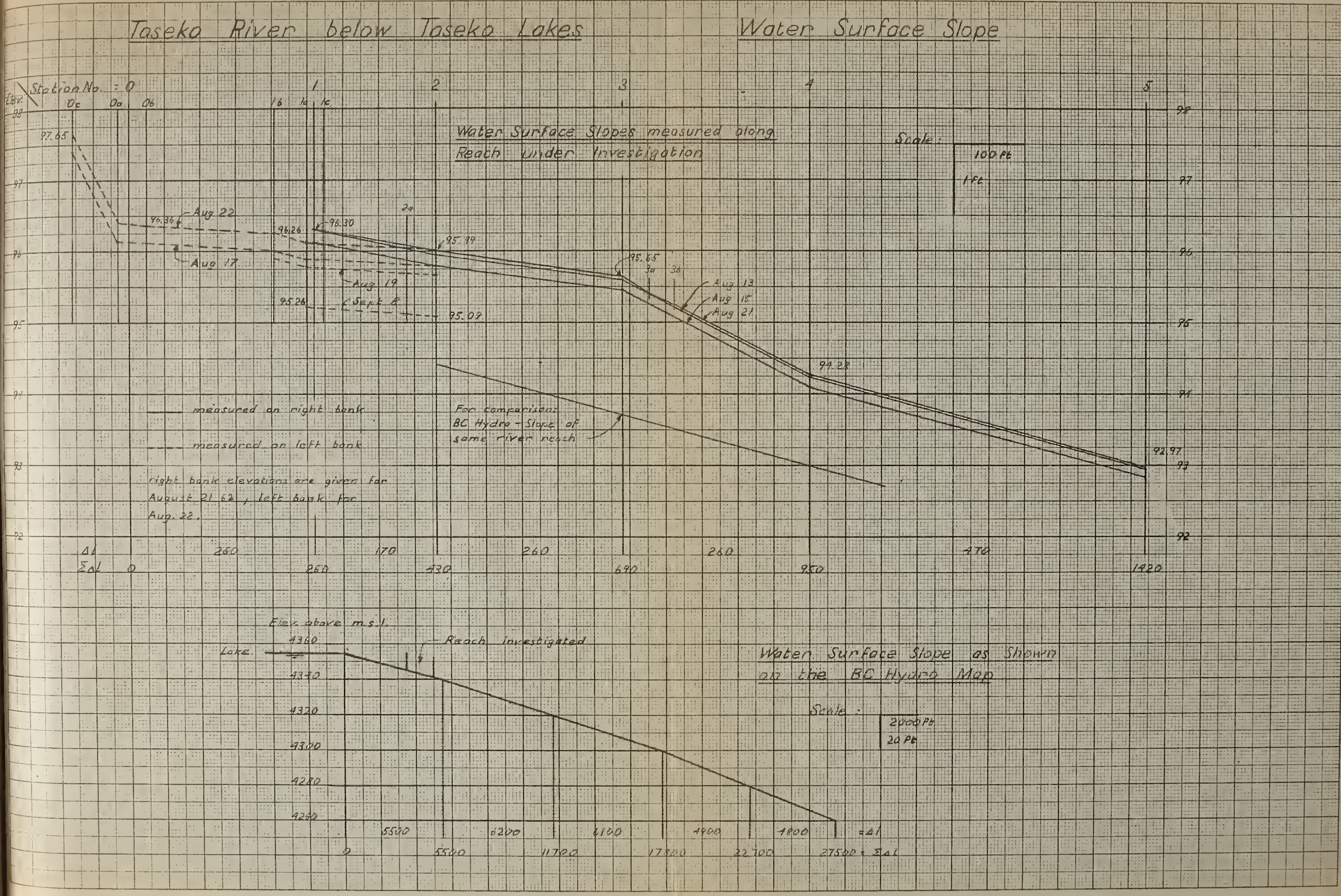

Toseko River below Toseko Lake
Sections
-50
left bank
0
50
100
150
200
right bank
250
Aug 13 Q = 3500 cfs
Sept. 8 Q = 2500 cfs
Scale : 10'
5'
measured
estimated
Station 1
100
90
BM 1 100.00 (Datum)
Aug. 13, 96.31
Sept. 8, 95.2
Veget.
BM 11, 96.65
High water
Station 2
100
90
BM 2 98.56
Trees
Aug. 13, 95.95
Sept. 8, 95.1
BM 22 97.65
high water mark, vegetation
Station 3
100
90
Trees
BM 3 96.45
Aug. 13, 95.60
Sept. 8, 94.8
old paved channel
Trees
Station 4
100
90
Vegetation
BM 4 94.71
Aug. 13, 60, 94.23
Sept. 8, 93.4
Trees
Station 5
100
90
BM 5 93.34
Trees
Aug. 13, 92.94
Sept. 8, 92.1
Trees

Fig. 27



Taseko River below Taseko Lakes
Water Surface Slope
Station No. 0
0c 0a 0b 1b 1a 1c 1 2 3 4 5
Elev.
Water Surface Slopes measured along Reach under Investigation
Scale: 100 ft 1 ft
97.65
96.36
Aug 22
Aug 17
96.26
96.30
95.94
95.65
Aug 19
Sept 8
95.26
95.09
Aug 13
Aug 15
Aug 21
94.22
92.97
measured on right bank
measured on left bank
For comparison: BC Hydro - Slope of same river reach
right bank elevations are given for August 21 62, left bank for Aug. 22.
Δl
Σ Δl
260 170 260 260 470
0 260 430 690 950 1420
Elev. above m.s.l.
Lake
Reach Investigated
4360 4340 4320 4300 4280 4260
Water Surface Slope as Shown on the BC Hydro Map
Scale: 2000 Ft 20 Ft
5500 6200 6100 4900 4800 = Δl
0 5500 11700 17800 22700 27500 = Σ Δl

Fig. 28

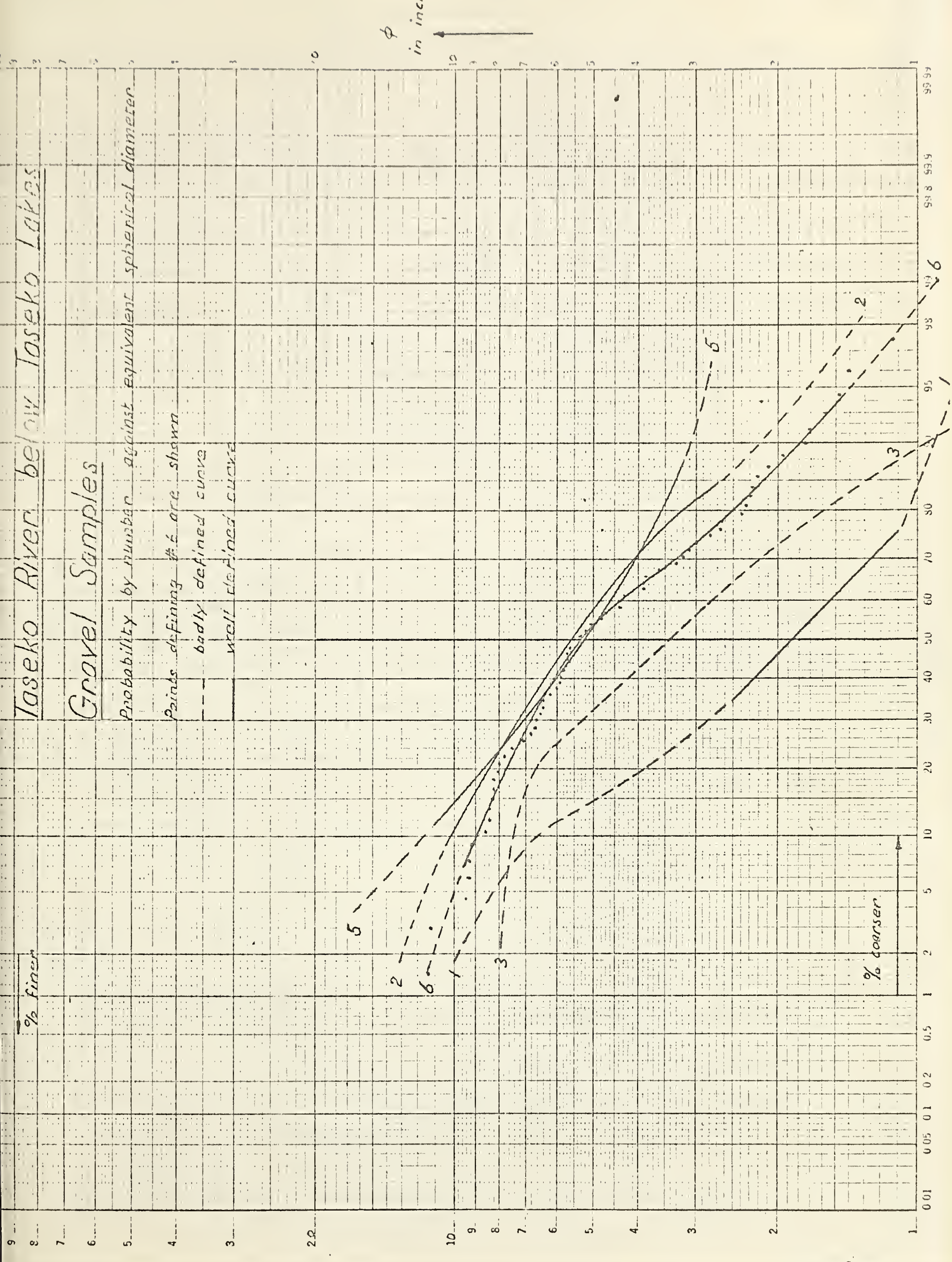

Taseko River below Taseko Lakes
Gravel Samples
Probability by number against equivalent spherical diameter
Points defining #1 are shown
badly defined curve
well defined curve
% finer
% coarser
ϕ in inches

Fig. 29

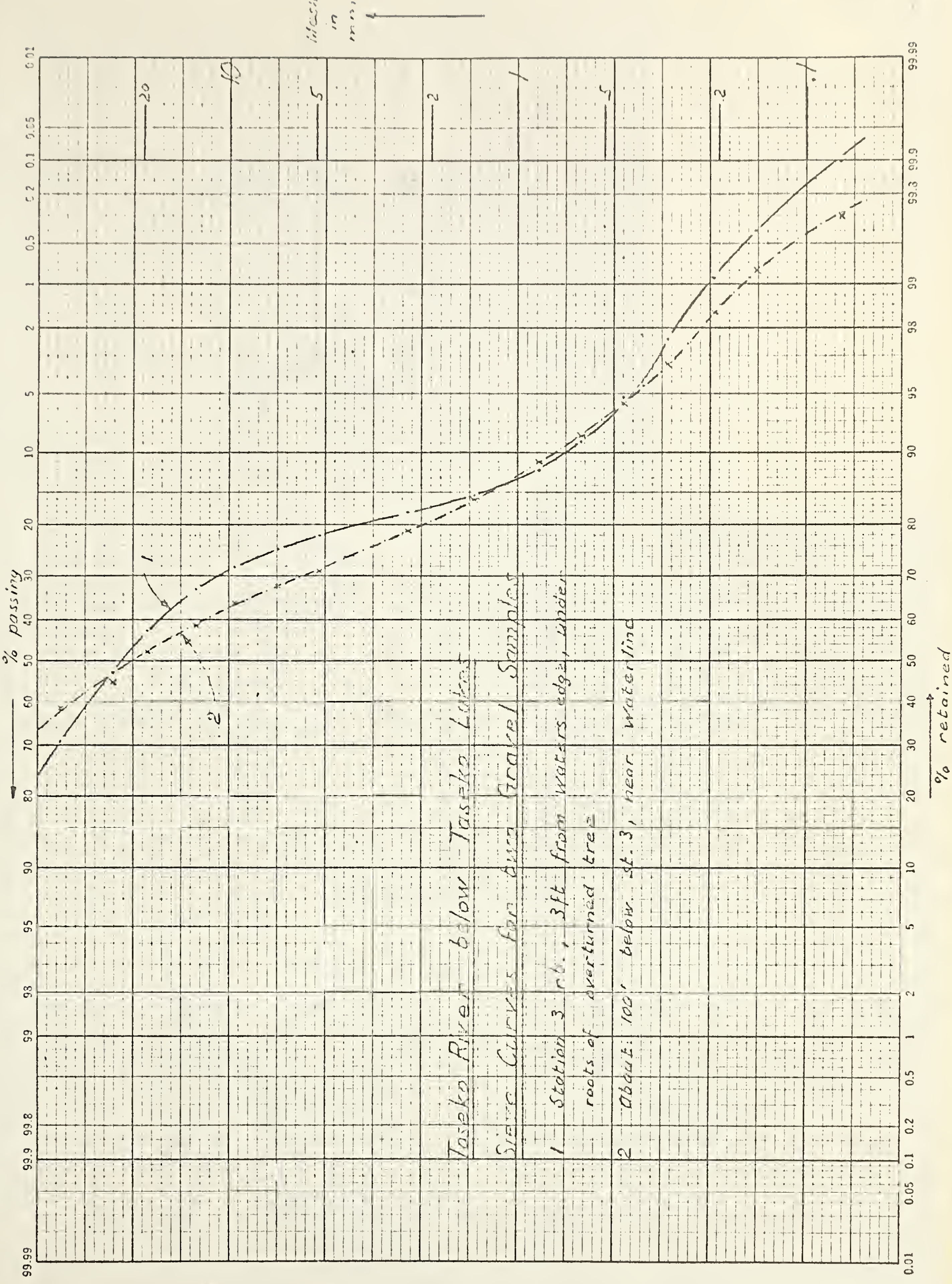
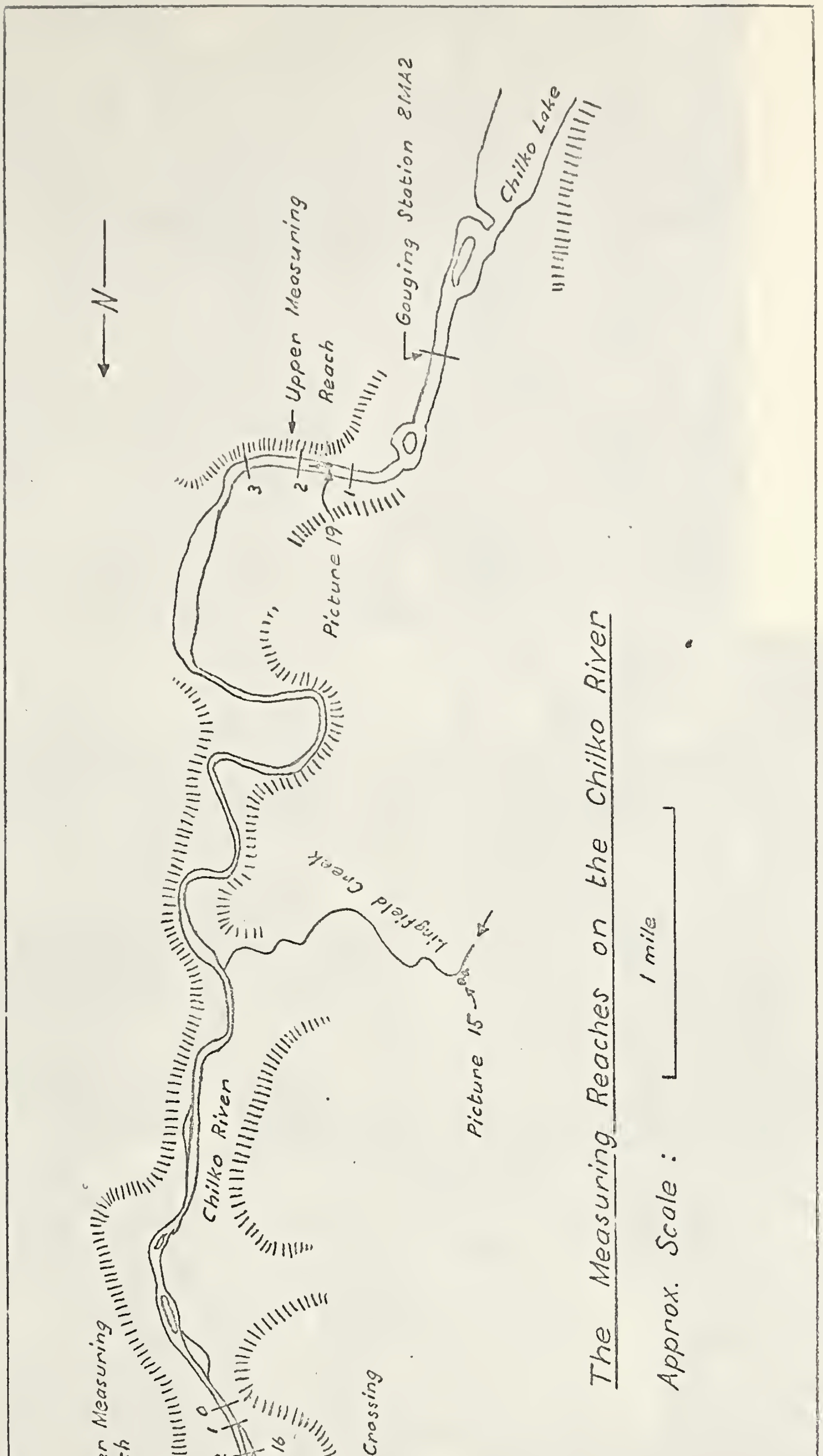

Mesh in mm
% passing
% retained
Taseko River below Taseko Lakes
Sieve Curves for two Gravel Samples
1 Station 3 r.b., 3 ft from waters edge, under roots of overturned tree
2 About 100' below St. 3, near waterline

Fig. 30



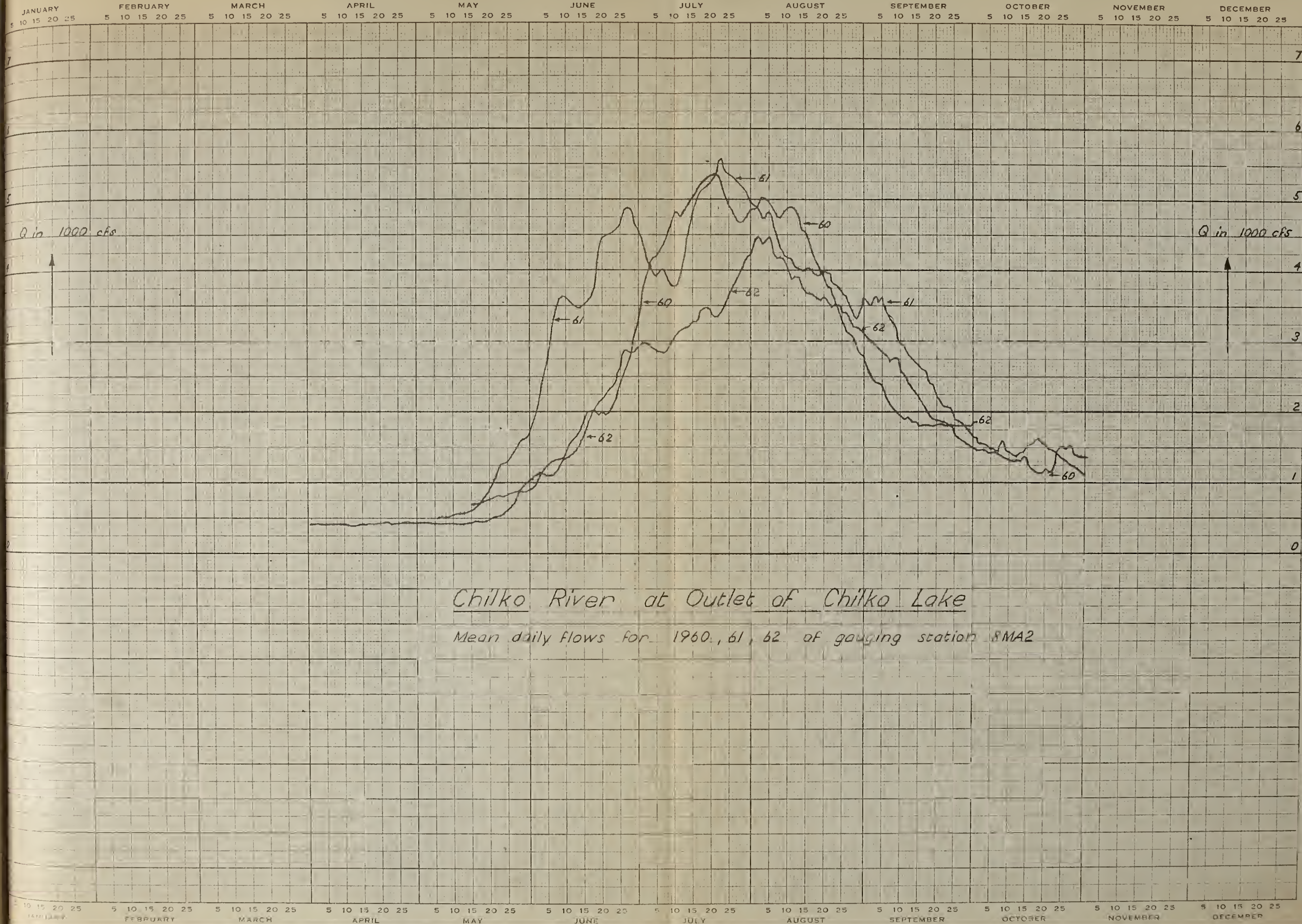
The Measuring Reaches on the Chilko River

Fig. 31

CHILKO RIVER AT HENRY'S CROSSING

APPROX. SCALE I INCH 2450 FEET

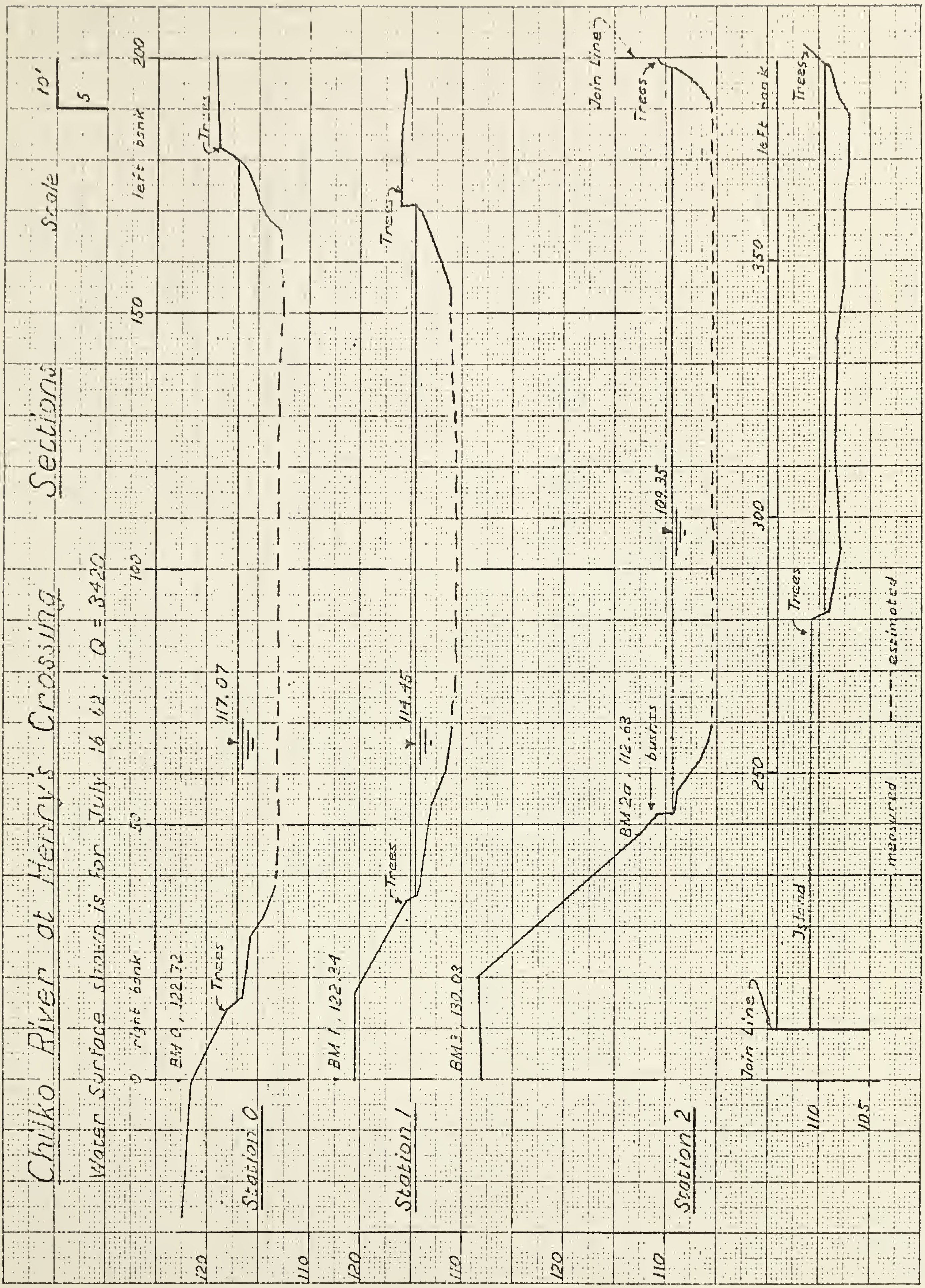
Fig 32



Chilko River at Outlet of Chilko Lake

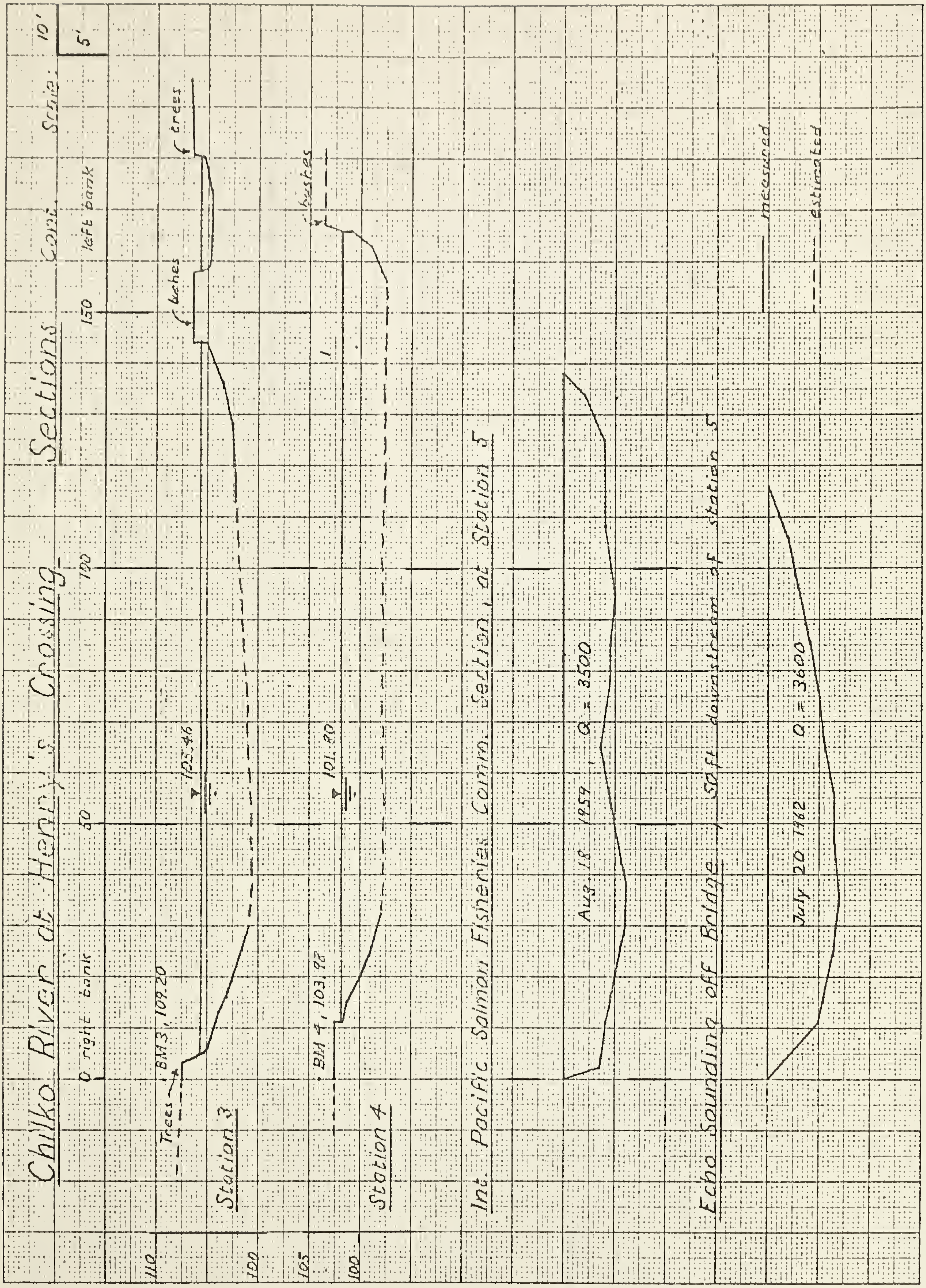
Mean daily flows for 1960, 61, 62 of gauging station 8MA2

Fig. 33



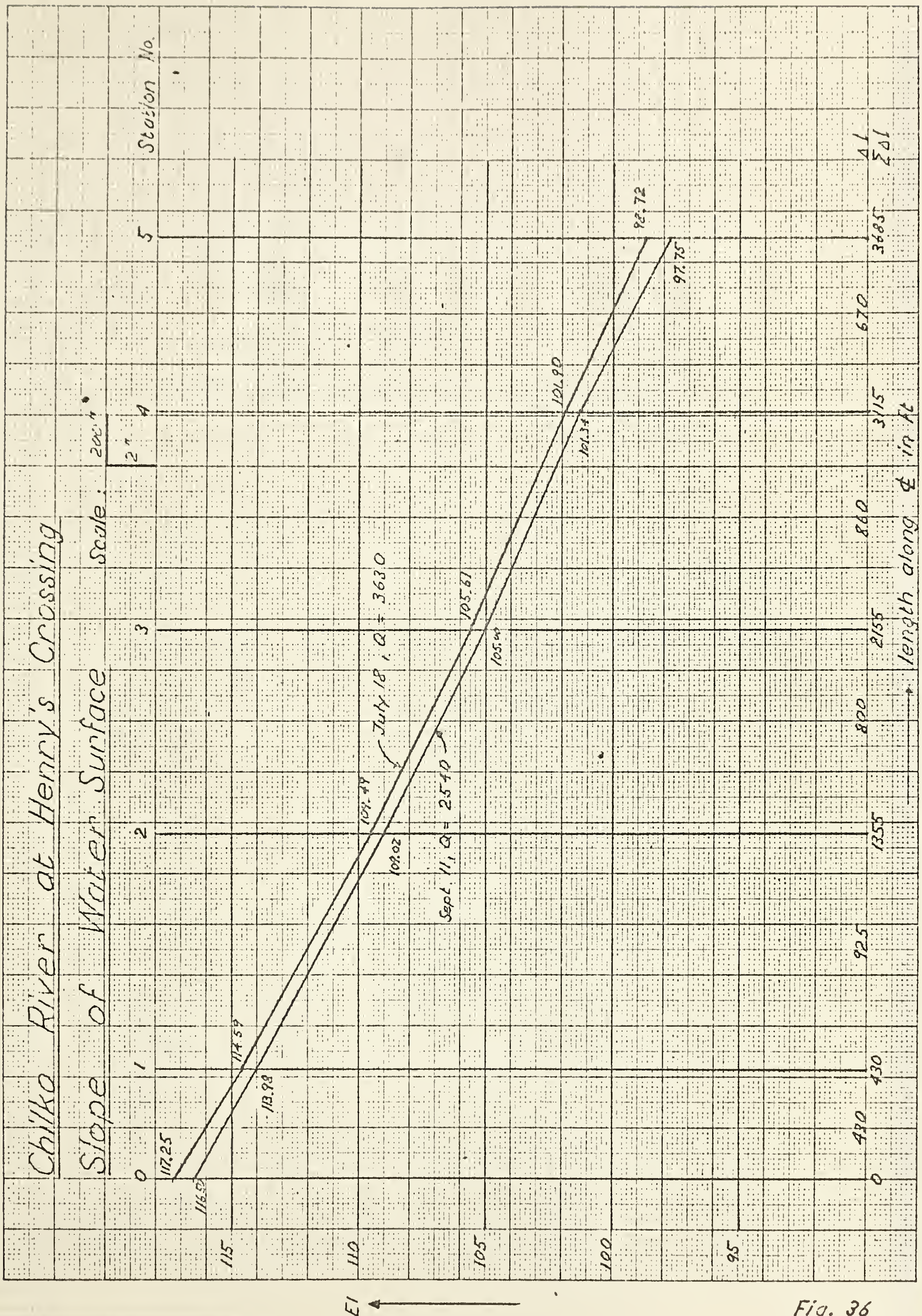
Chilko River at Henry's Crossing
Sections
Water Surface shown is for July 16 62, Q = 3420
Scale
10'
5
Station 0
Station 1
Station 2
BM 0, 122.72
BM 1, 122.34
BM 3, 130.03
BM 2a, 112.63
117.07
114.45
109.35
right bank
left bank
Trees
Join Line
bushes
Island
measured
estimated
0
50
100
150
200
250
300
350
120
110
105

Fig. 34



Chilko River at Henry's Crossing Sections Cont.
Scale: 10'
5'
0 right bank
50
100
150
left bank
trees
bushes
Trees
BM 3, 109.20
105.46
Station 3
bushes
BM 4, 103.98
101.20
Station 4
Int. Pacific Salmon Fisheries Comm. Section, at Station 5
Aug. 18 1959, Q = 3500
Echo Sounding off Bridge, 50 ft downstream of station 5
July 20 1962 Q = 3600
measured
estimated
110
100
105
100

Fig. 35



Chilko River at Henry's Crossing
Slope of Water Surface
Scale: 2" = 200'
Station No.
0
1
2
3
4
5
117.25
116.50
114.59
113.98
109.49
109.02
105.61
105.00
101.90
101.34
98.72
97.75
July 18, Q = 3630
Sept. 11, Q = 2540
115
110
105
100
95
El
0
430
430
925
1355
800
2155
860
3115
670
3685
Δl
$\Sigma\Delta l$
length along ℄ in ft

Fig. 36

Water Surface Slope of the Chilko River as shown on a BC-Hydro map.

Scale: 1000′ / 10′

For comparison slope measurements by the author and by the Int. Pacific Salmon Fisheries Commission are also shown

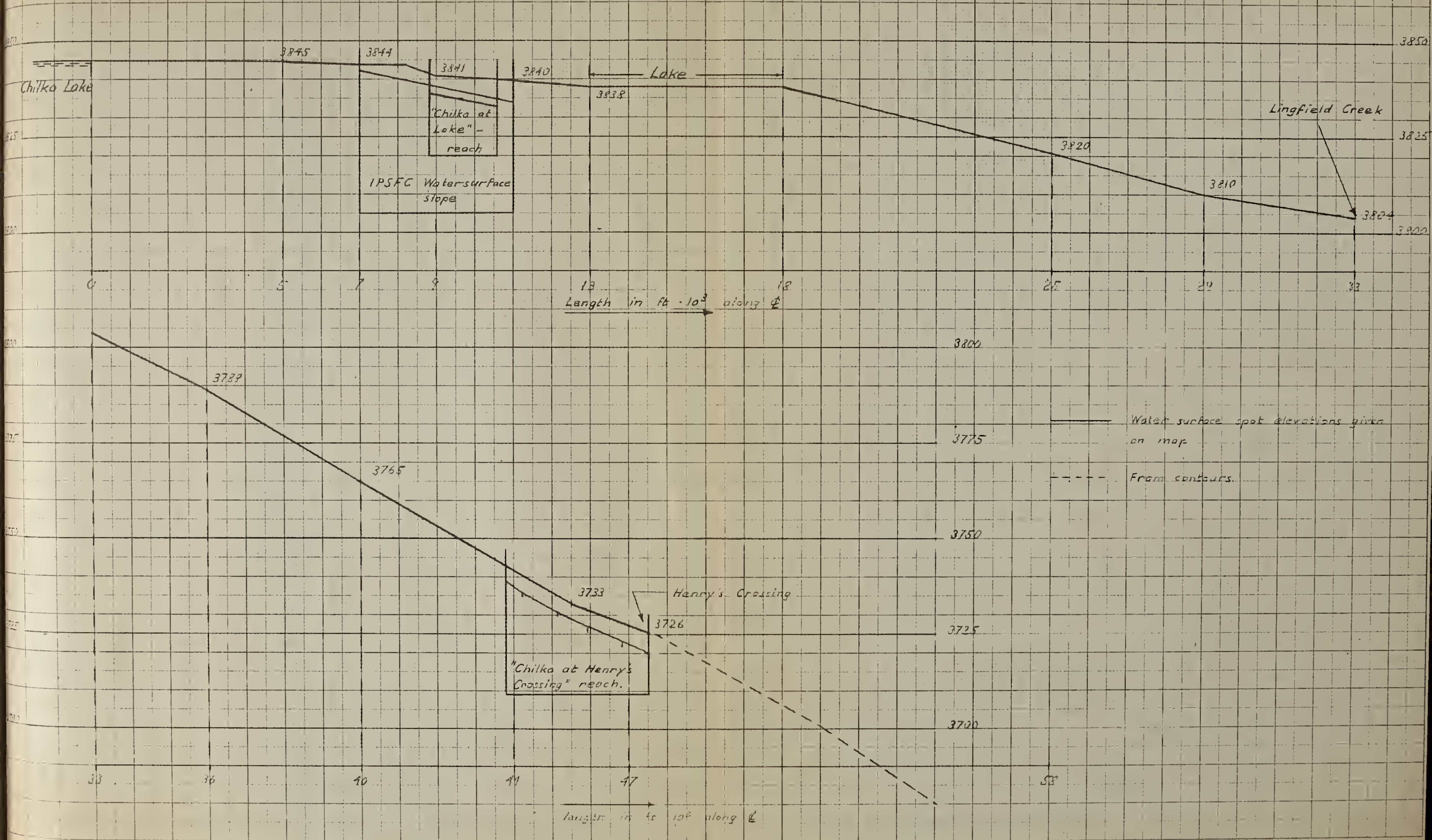


Fig. 37

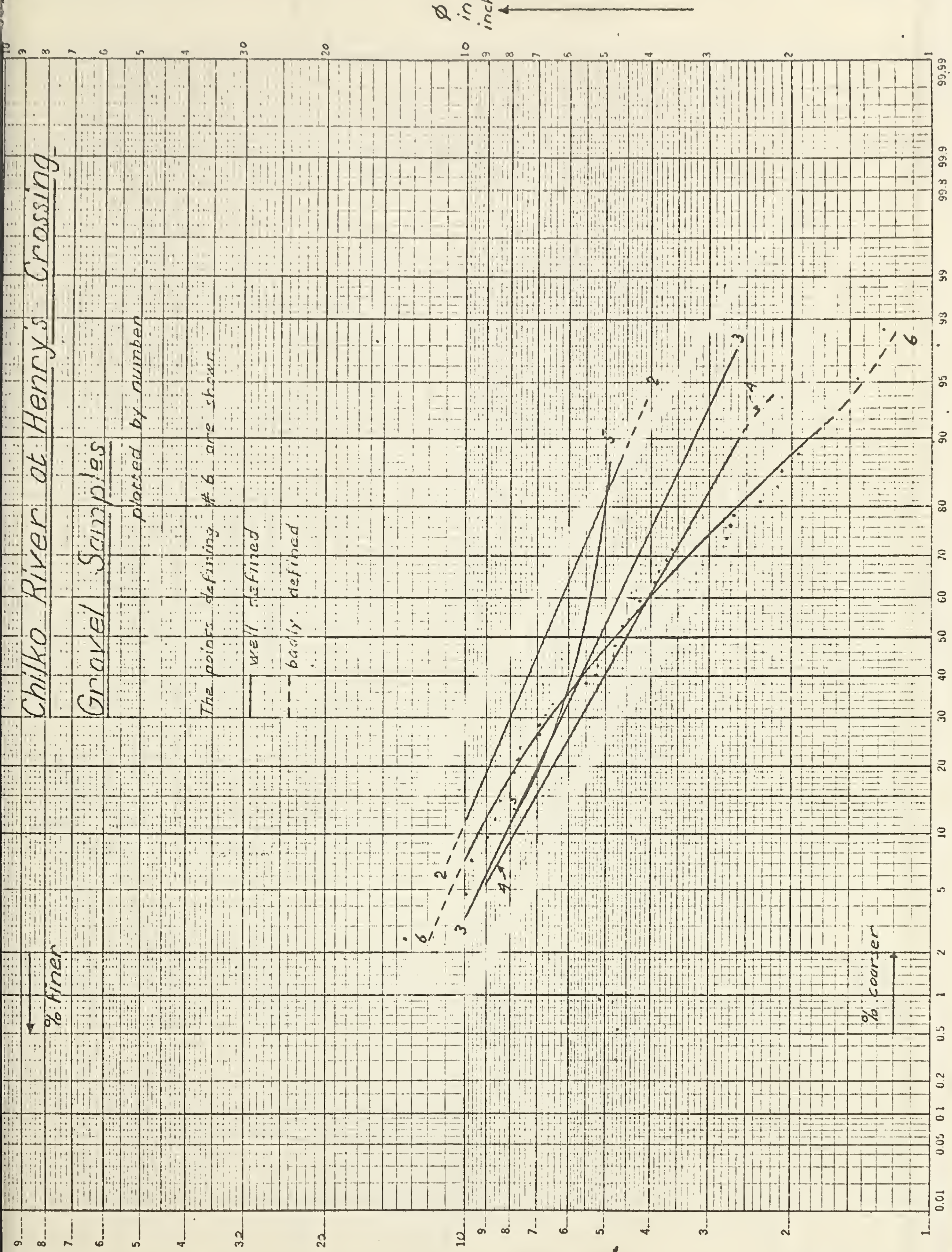

Chilko River at Henry's Crossing
Gravel Samples
plotted by number
The points defining # 6 are shown
well defined
badly defined
% finer
% coarser
Ø in inches

Fig. 38

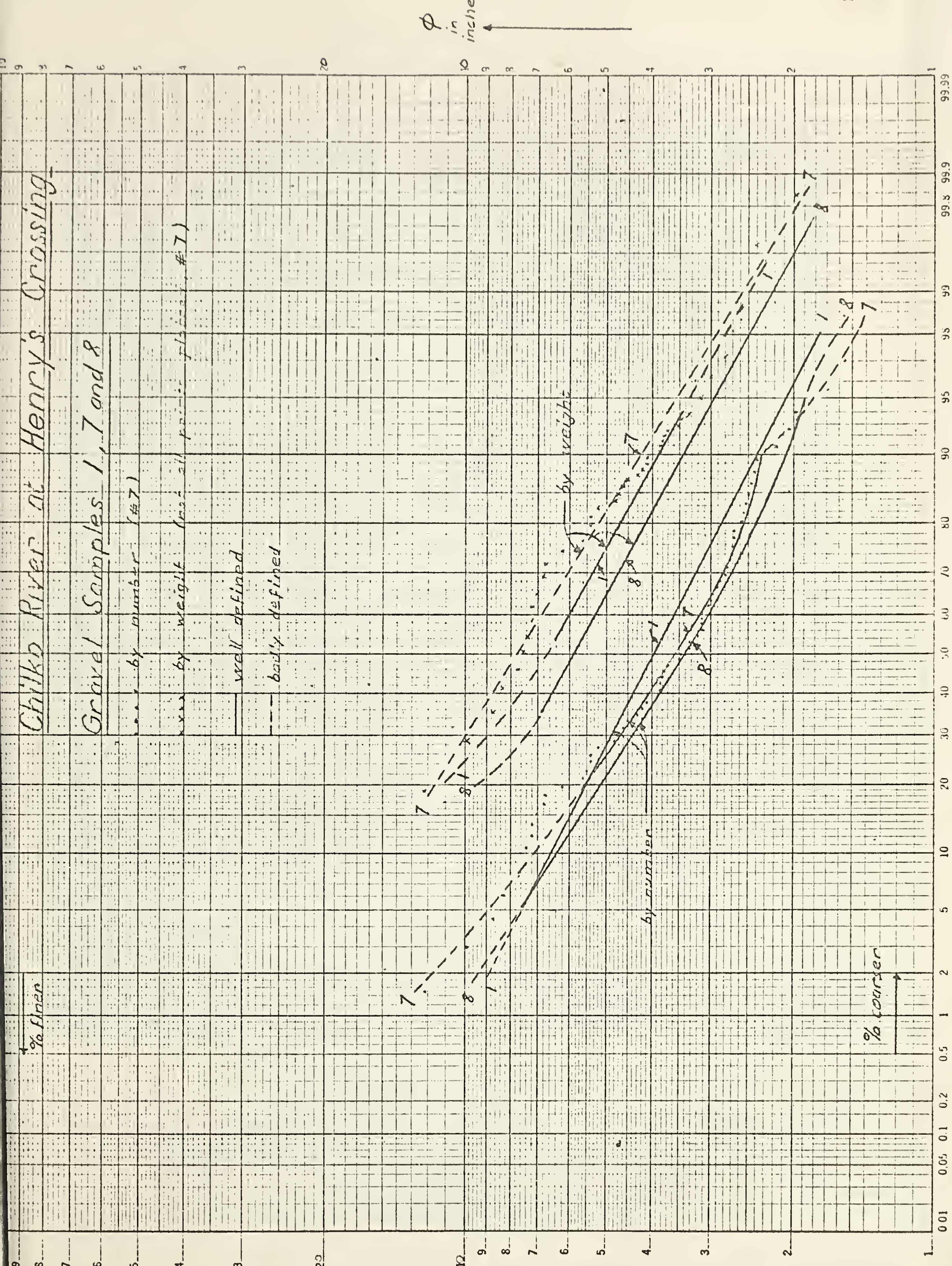

Chilko River at Henry's Crossing
Gravel Samples 1, 7 and 8
by number (#7)
by weight
well defined
badly defined
d in inches
% finer
% coarser
by weight
by number

Fig. 39

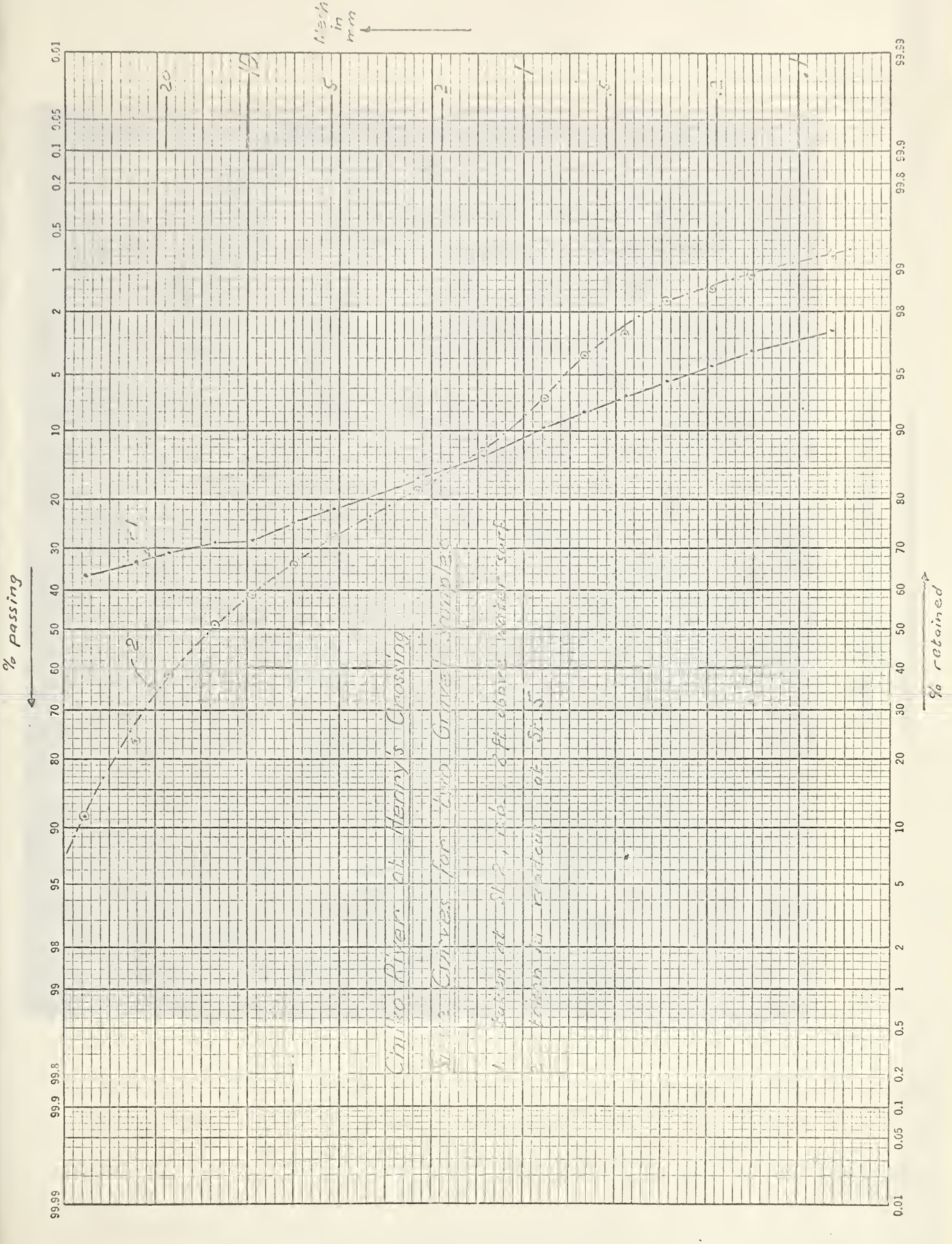

Mesh in mm
% passing
% retained
Chilko River at Henry's Crossing

Fig. 40

THE OUTLET OF CHILKO LAKE

APPROX. SCALE: 1 INCH = 1800 FEET

FIG. 41

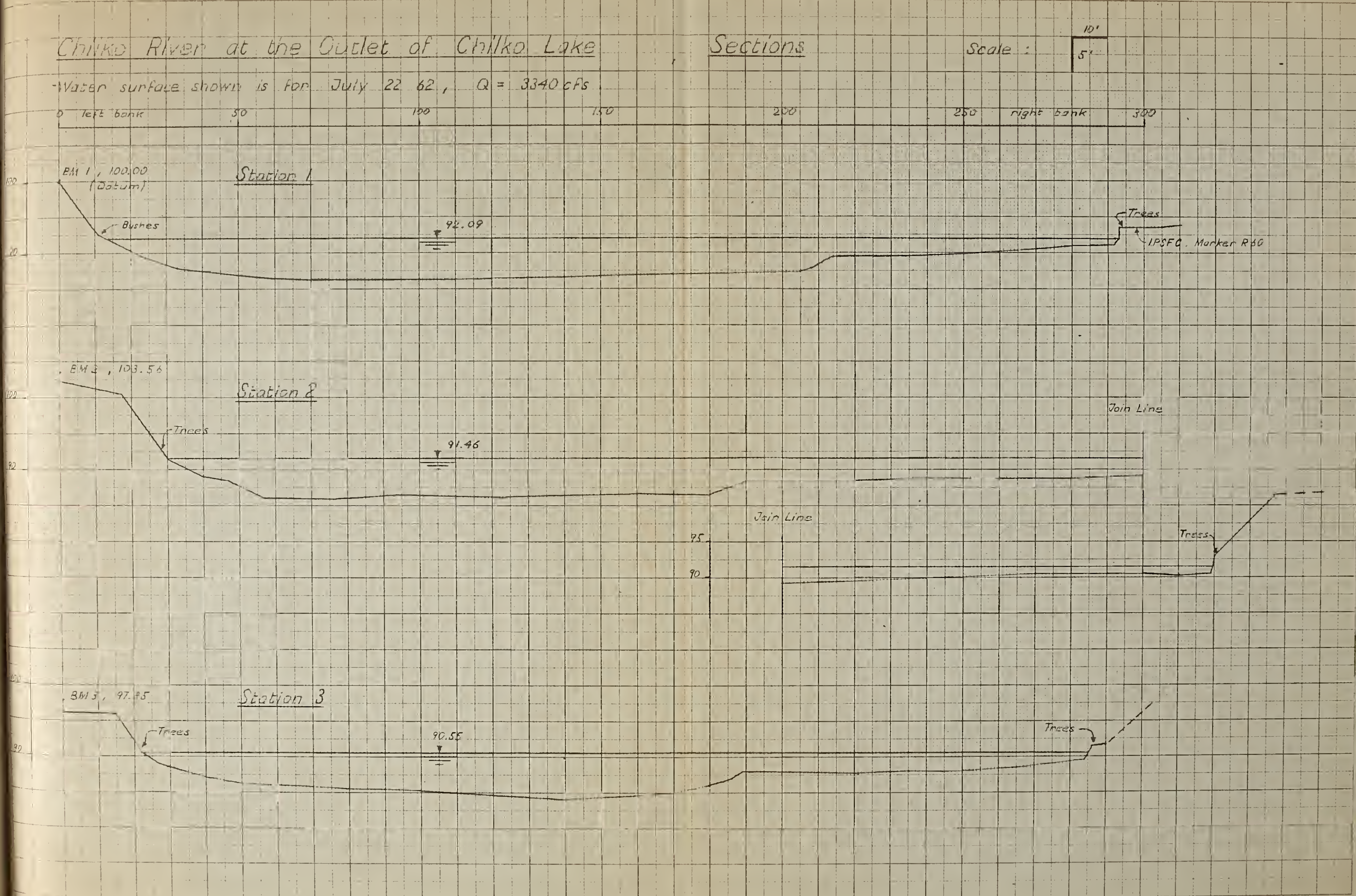

Chilko River at the Outlet of Chilko Lake
Sections
Scale :
10'
5'
Water surface shown is for July 22 62 , Q = 3340 cfs
0 left bank
50
100
150
200
250 right bank
300
Station 1
BM 1 , 100.00 (Datum)
Bushes
92.09
Trees
IPSFC Marker R80
100
90
Station 2
BM 2 , 103.56
Trees
91.46
Join Line
Join Line
95
90
Trees
100
90
Station 3
BM 3 , 97.85
Trees
90.55
Trees
100
90

Fig. 42

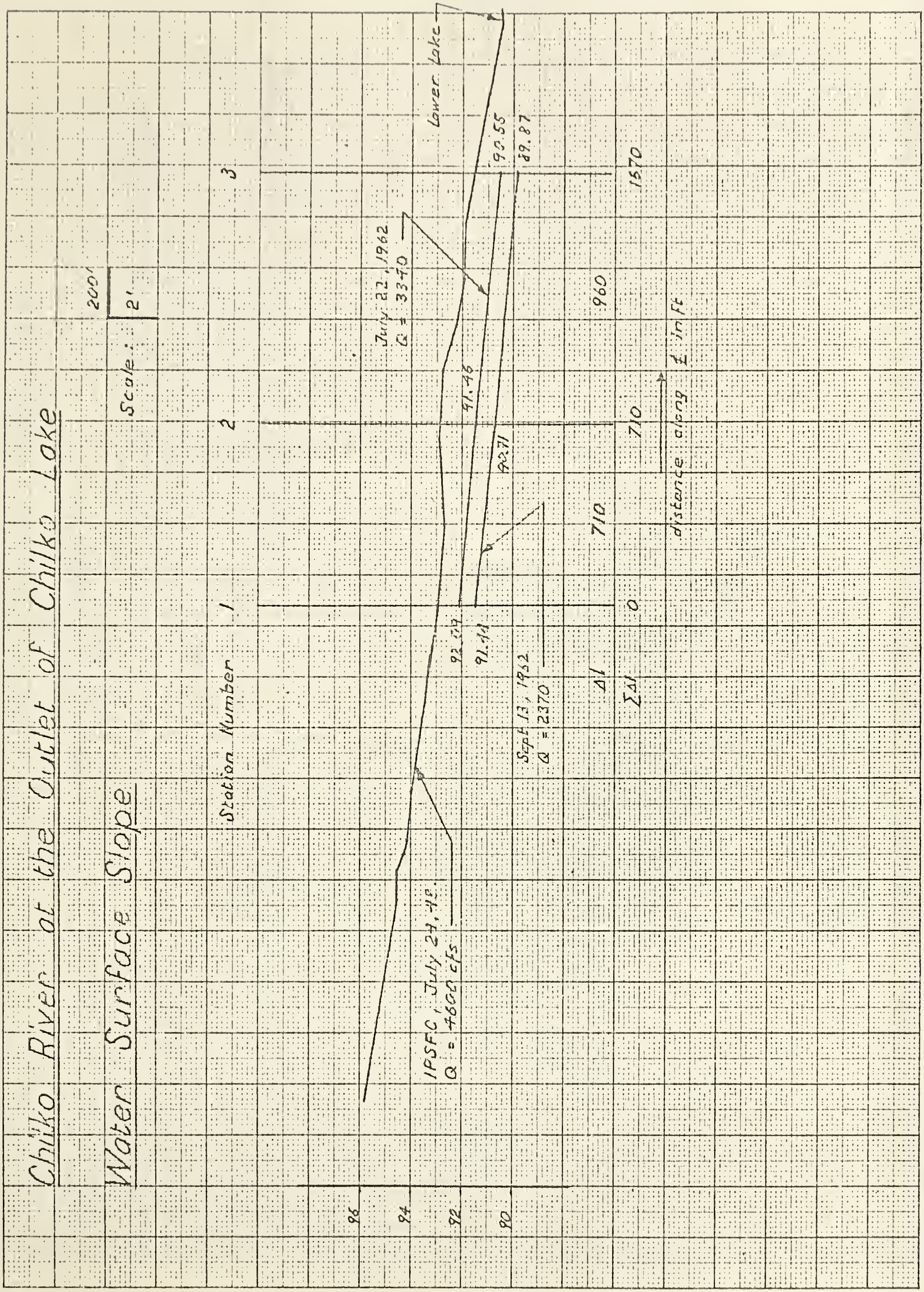

Chilko River at the Outlet of Chilko Lake
Water Surface Slope
Scale:
200'
2'
Station Number
1
2
3
Lower Lake
89.87
July 22, 1962
Q = 3340
IPSFC, July 24, 48
Q = 4600 cfs
Sept 13, 1962
Q = 2370
91.46
710
960
710
0
710
1570
distance along ℄ in ft
96
94
92
90

Fig. 43

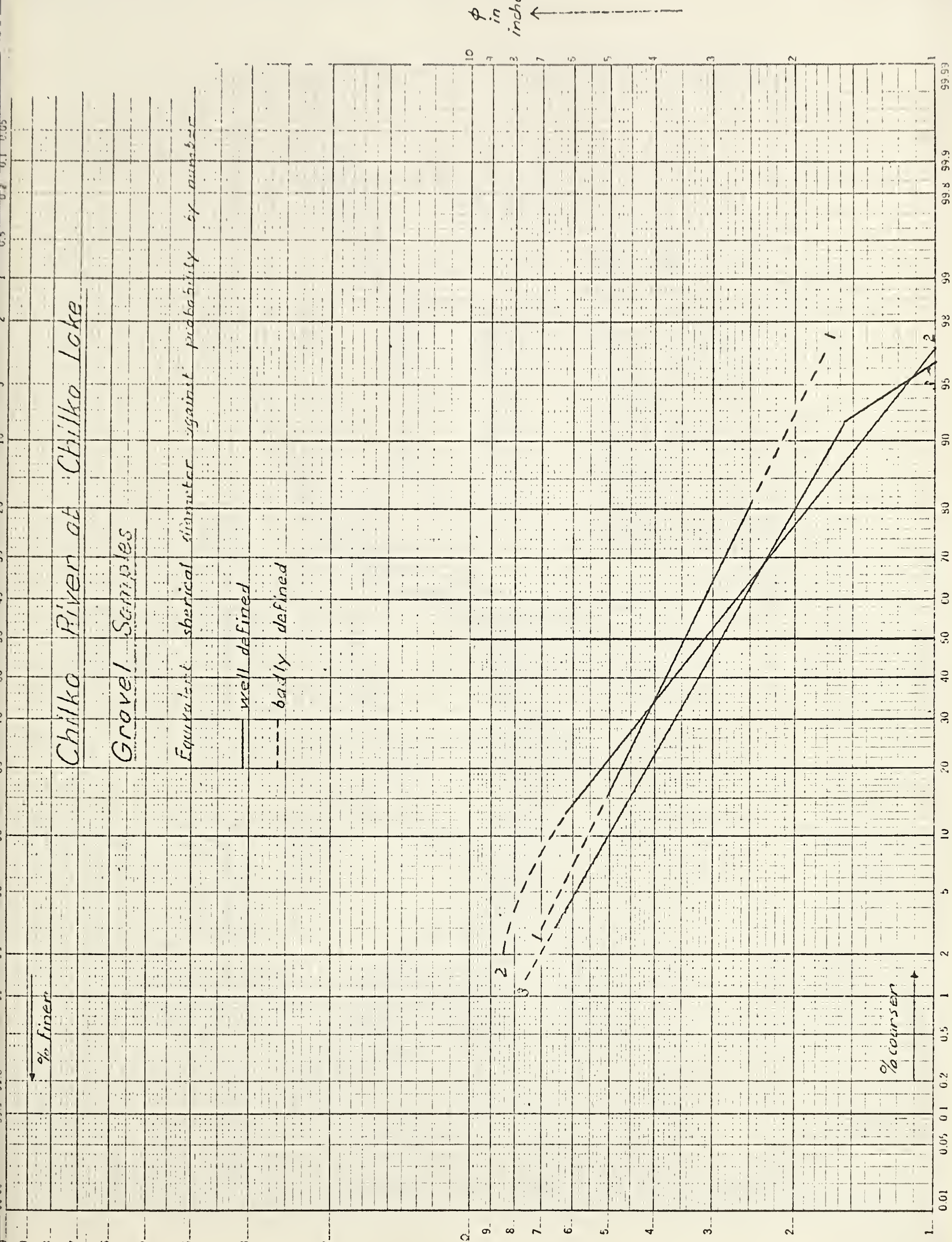

φ in inches
Chilko River at Chilko Lake
Gravel Samples
Equivalent spherical diameter against probability by number
well defined
badly defined
% finer
% courser

Fig. 44

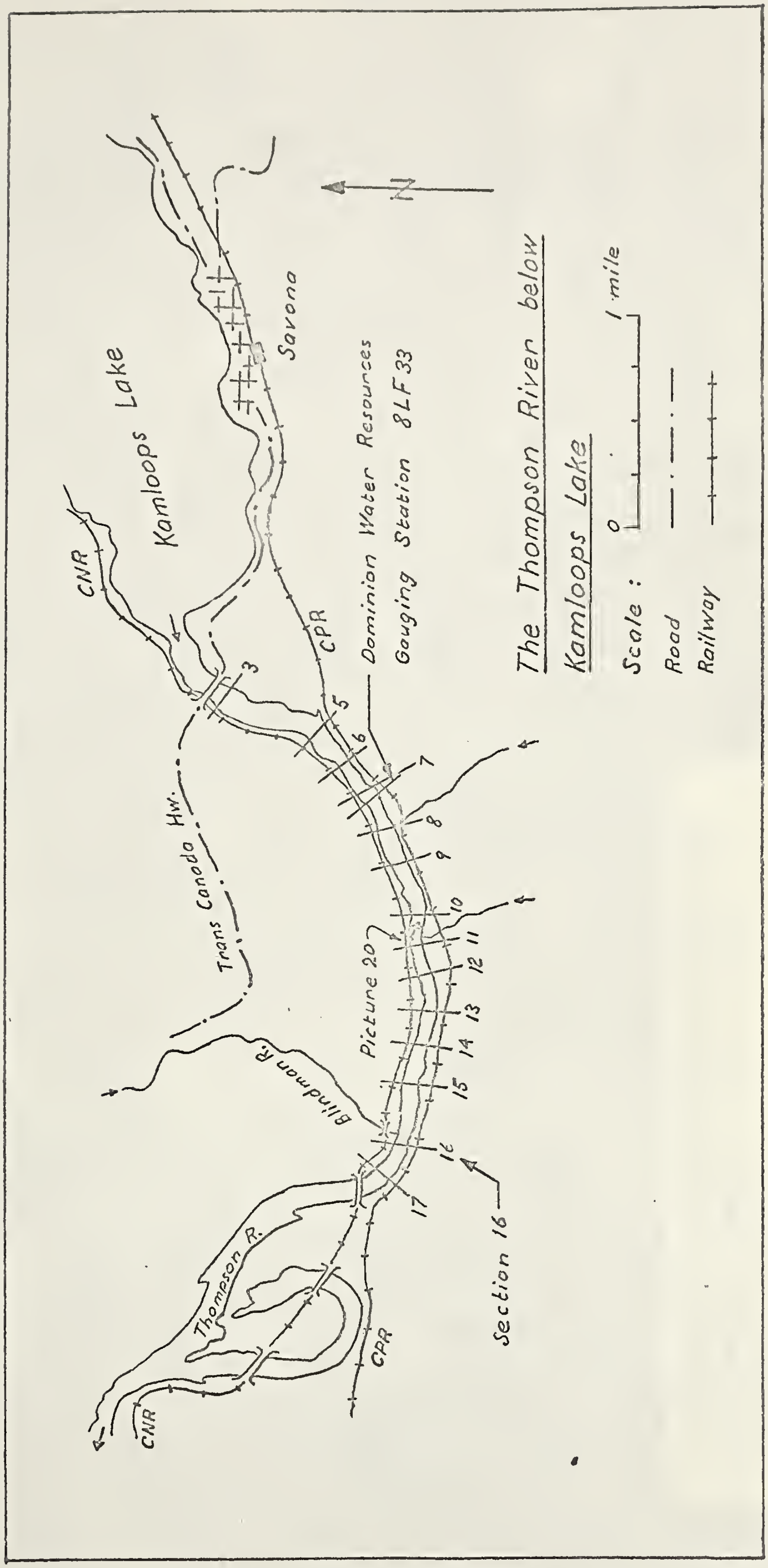
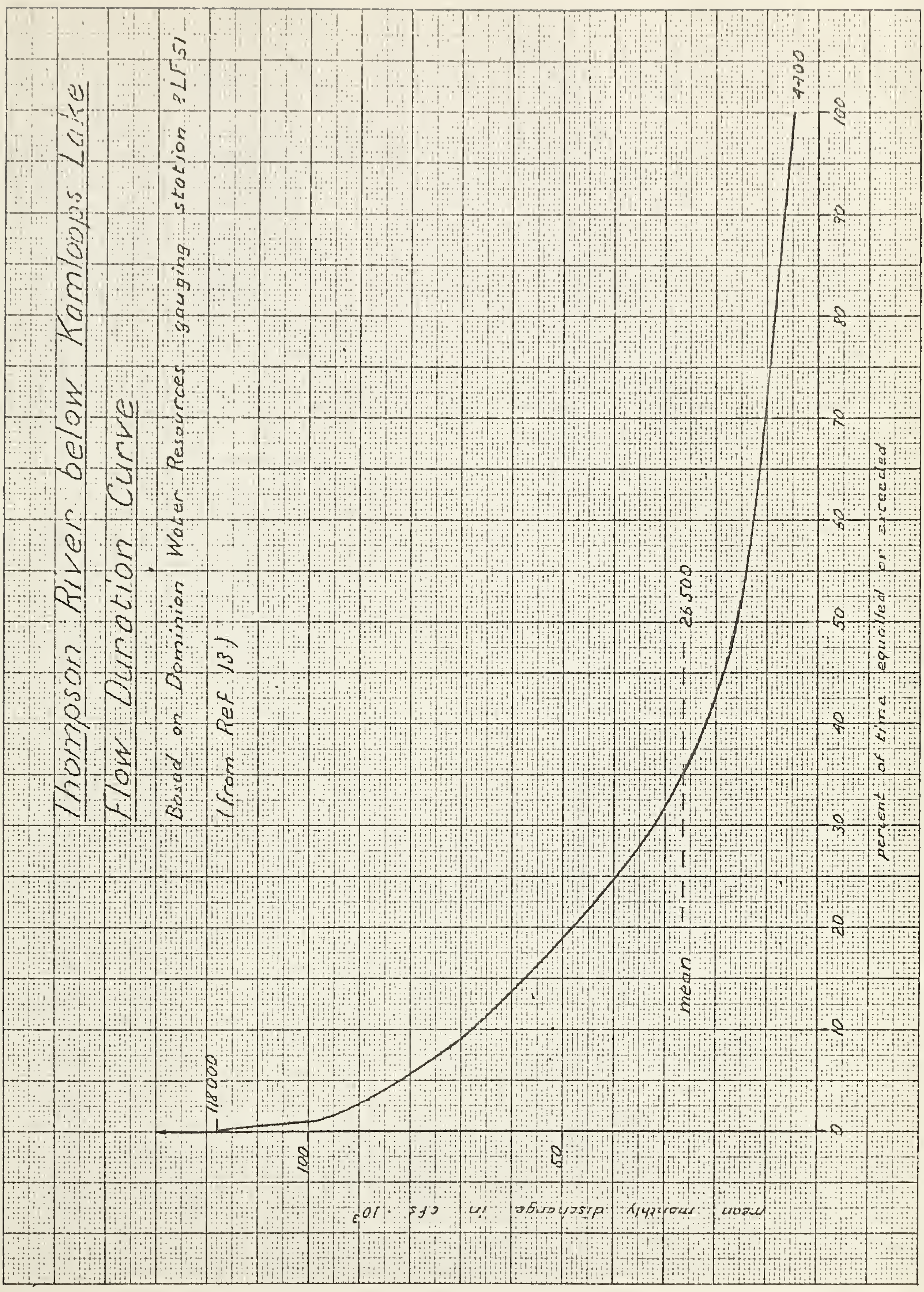

The Thompson River below Kamloops Lake
Scale : 0 1 mile
Road
Railway
Kamloops Lake
Savona
CNR
CPR
Dominion Water Resources Gauging Station 8LF33
Trans Canada Hw.
Picture 20
Blindman R.
Thompson R.
Section 16
CPR
CNR
3
5
6
7
8
9
10
11
12
13
14
15
16
17

Fig. 45

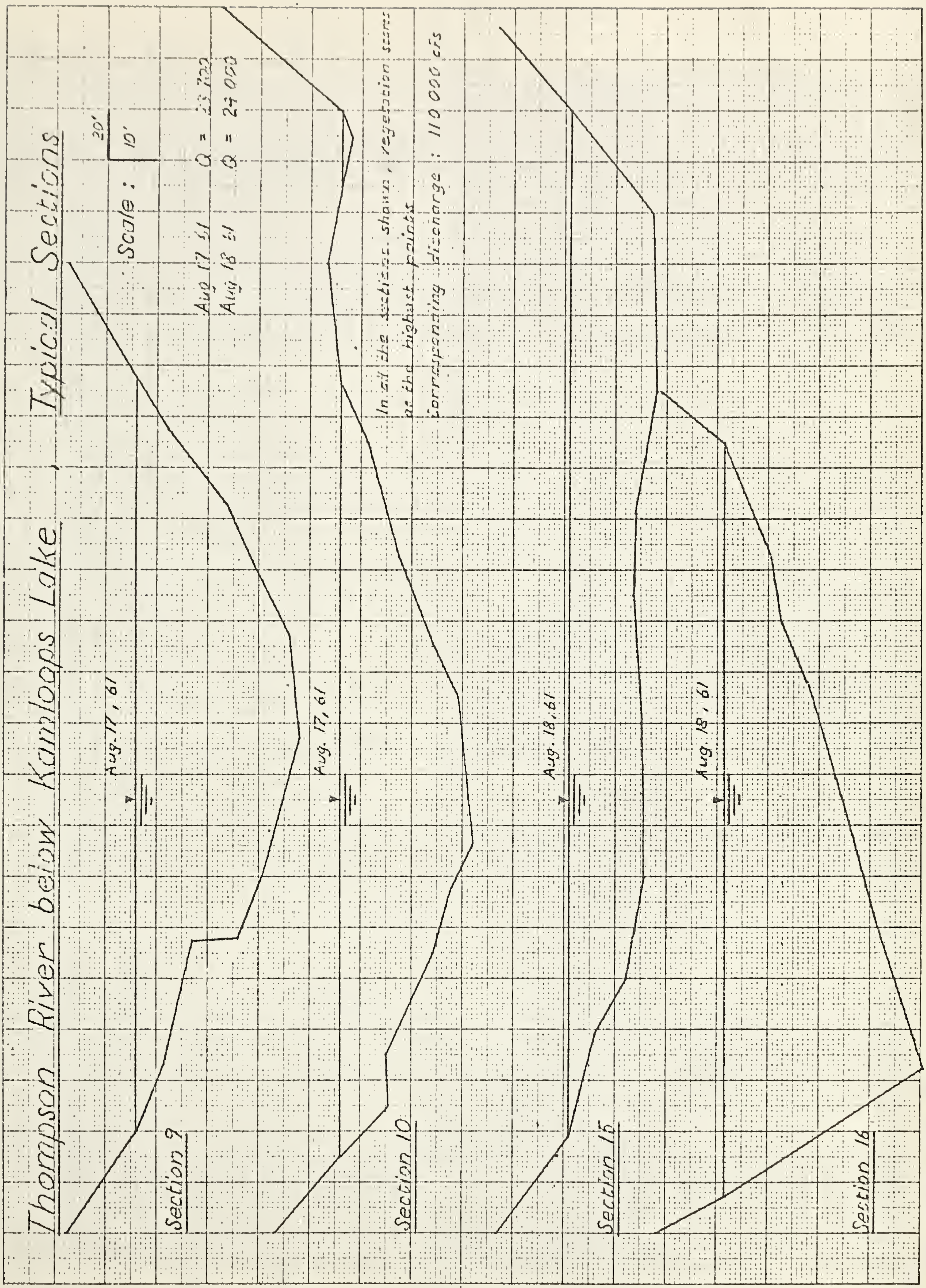

THOMPSON RIVER BELOW KAMLOOPS LAKE
AIR PHOTOS TAKEN ON JULY 21 1959
$Q_{THOMPSON}$ 61900 CFS.
APPROX. SCALE : 1 INCH = 1400 FEET
8
9
10
11
12
13
14

FIG. 46



Thompson River below Kamloops Lake
Flow Duration Curve
Based on Dominion Water Resources gauging station 2LF51
(from Ref 13.)
118000
4700
mean 26 500
100
50
mean monthly discharge in cfs. 10^3
0
10
20
30
40
50
60
70
80
90
100
percent of time equalled or exceeded

Fig 47



Thompson River below Kamloops Lake, Typical Sections
Scale :
20'
10'
Aug 18 61 Q = 24 000
Corresponding discharge : 110 000 cfs
Aug. 17, 61
Aug. 17, 61
Aug. 18, 61
Aug. 18, 61
Section 9
Section 10
Section 15
Section 16

Fig. 48

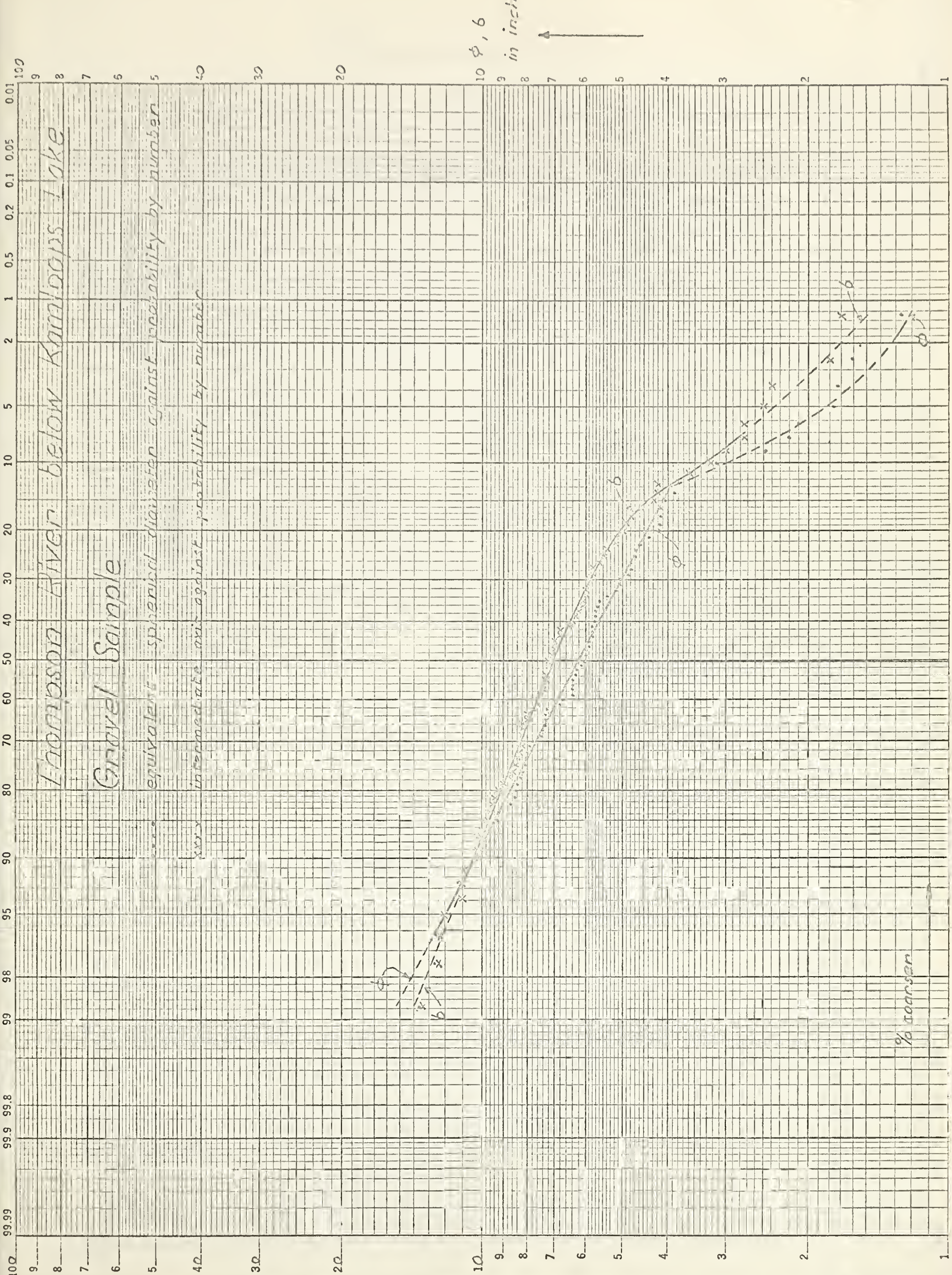

Thompson River below Kamloops Lake
Gravel Sample
equivalent spherical diameter against probability by number
intermediate axis against probability by number
d, b in inch
% coarser

Fig. 49

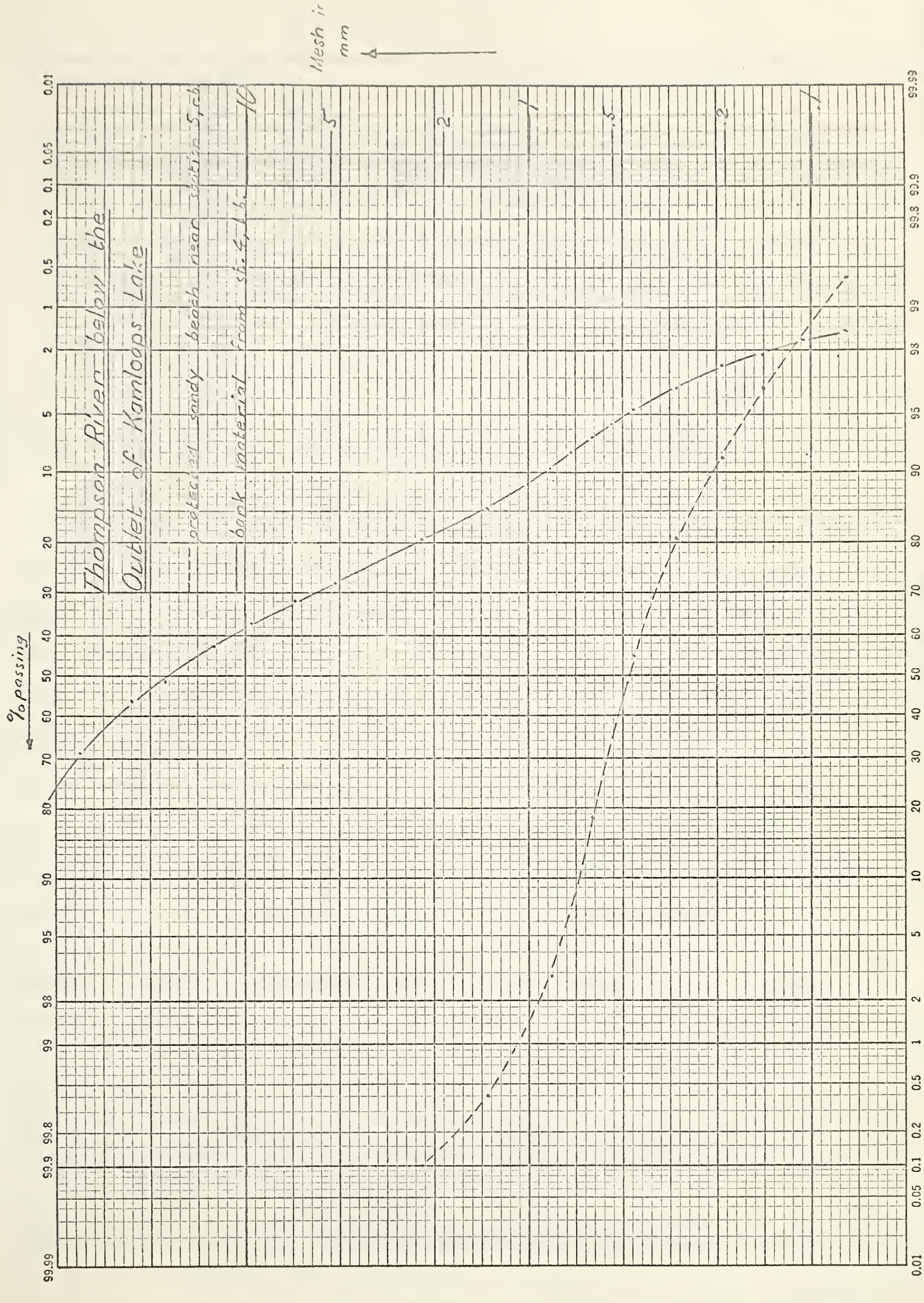

Thompson River below the
Outlet of Kamloops Lake
protected sandy beach near section 5, cb
bank material from st. 4, l.b.
Mesh in mm
% passing
% retained

Fig. 50

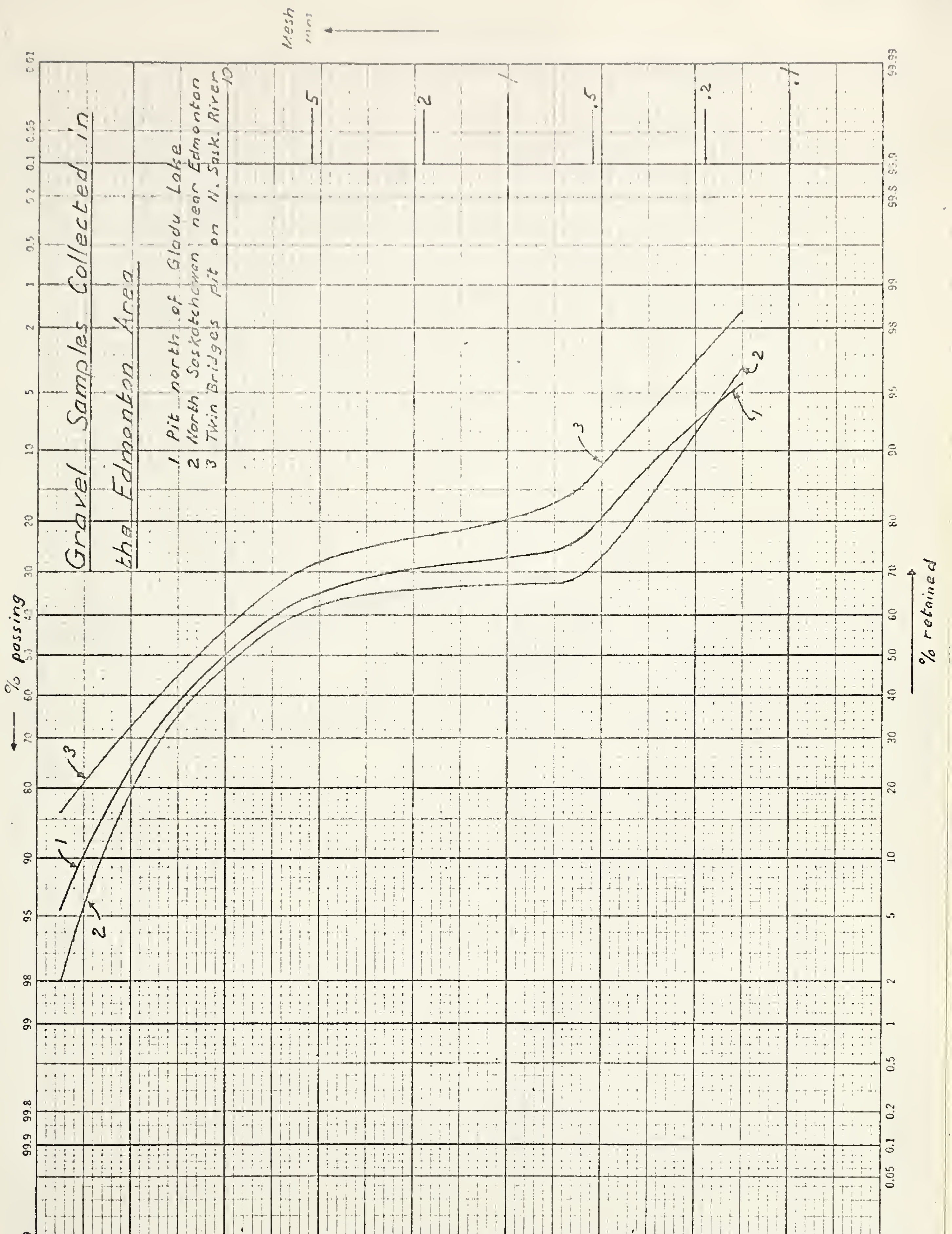

Gravel Samples Collected in the Edmonton Area
1 Pit north of Glodu Lake
2 North Soskatchewan near Edmonton
3 Twin Bridges pit on N. Sask. River
% passing
% retained
Mesh
mm
10
5
2
1
.5
.2
.1

Fig. 51

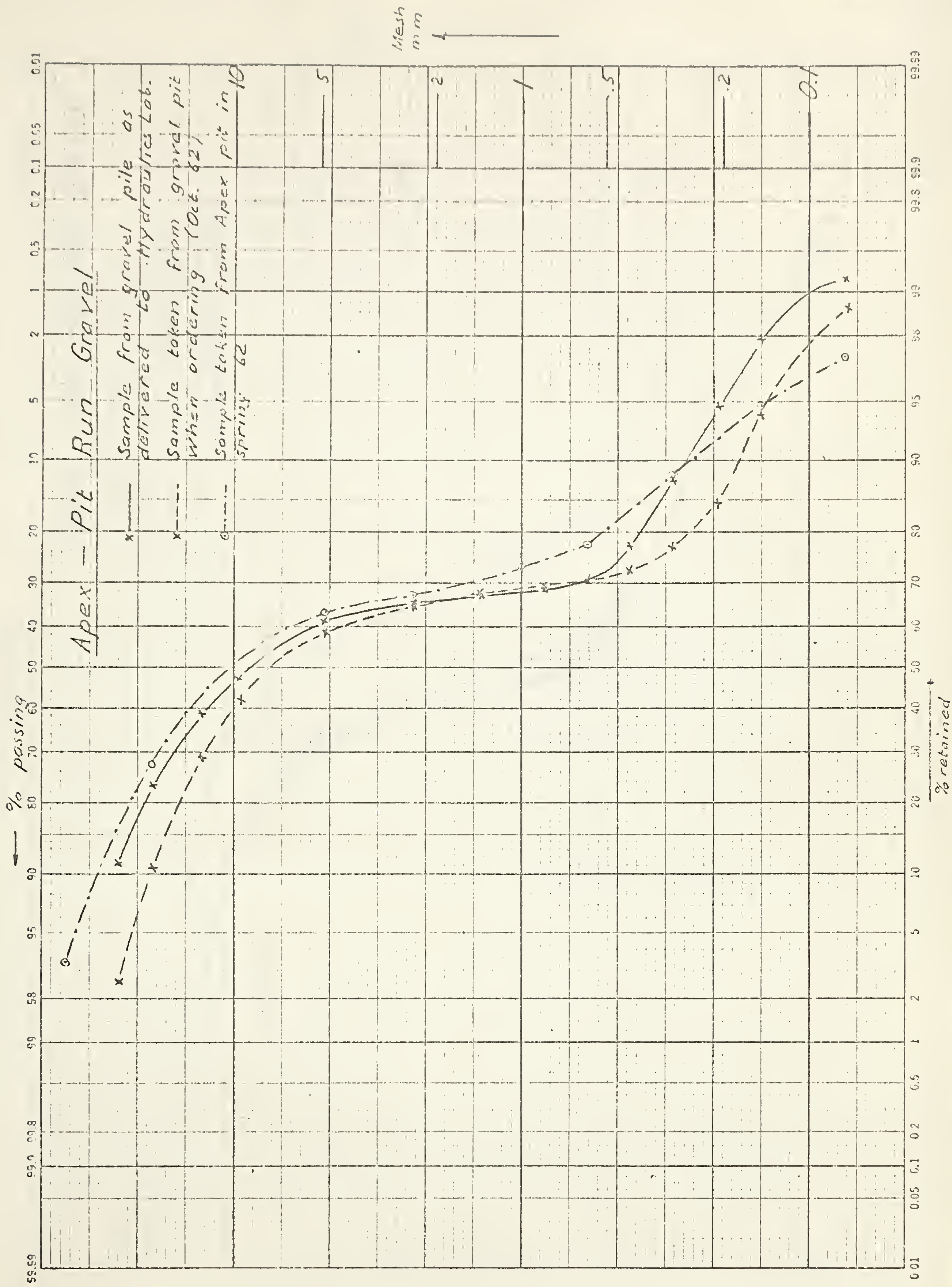

Apex – Pit Run Gravel
Sample from gravel pile as delivered to Hydraulics Lab.
Sample taken from gravel pit when ordering (Oct. 62)
Sample taken from Apex pit in spring 62
% passing
% retained
Mesh mm
10
5
2
1
.5
.2
0.1

Fig 52

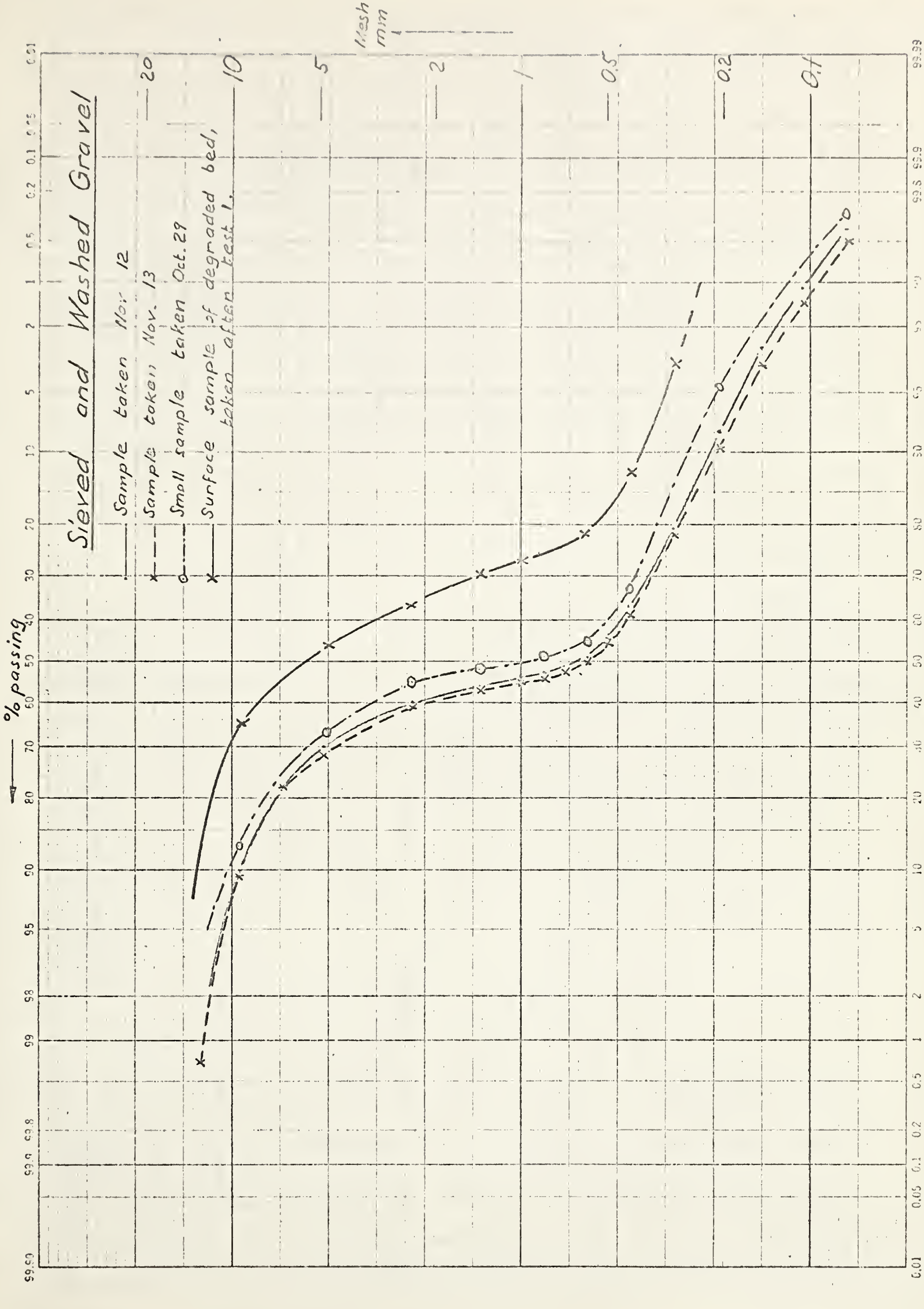

Sieved and Washed Gravel
Sample taken Nov 12
Sample taken Nov. 13
Small sample taken Oct. 29
Surface sample of degraded bed, taken after test I.
% passing
% retained
Mesh in mm
20
10
5
2
1
0.5
0.2
0.1

Fig. 53

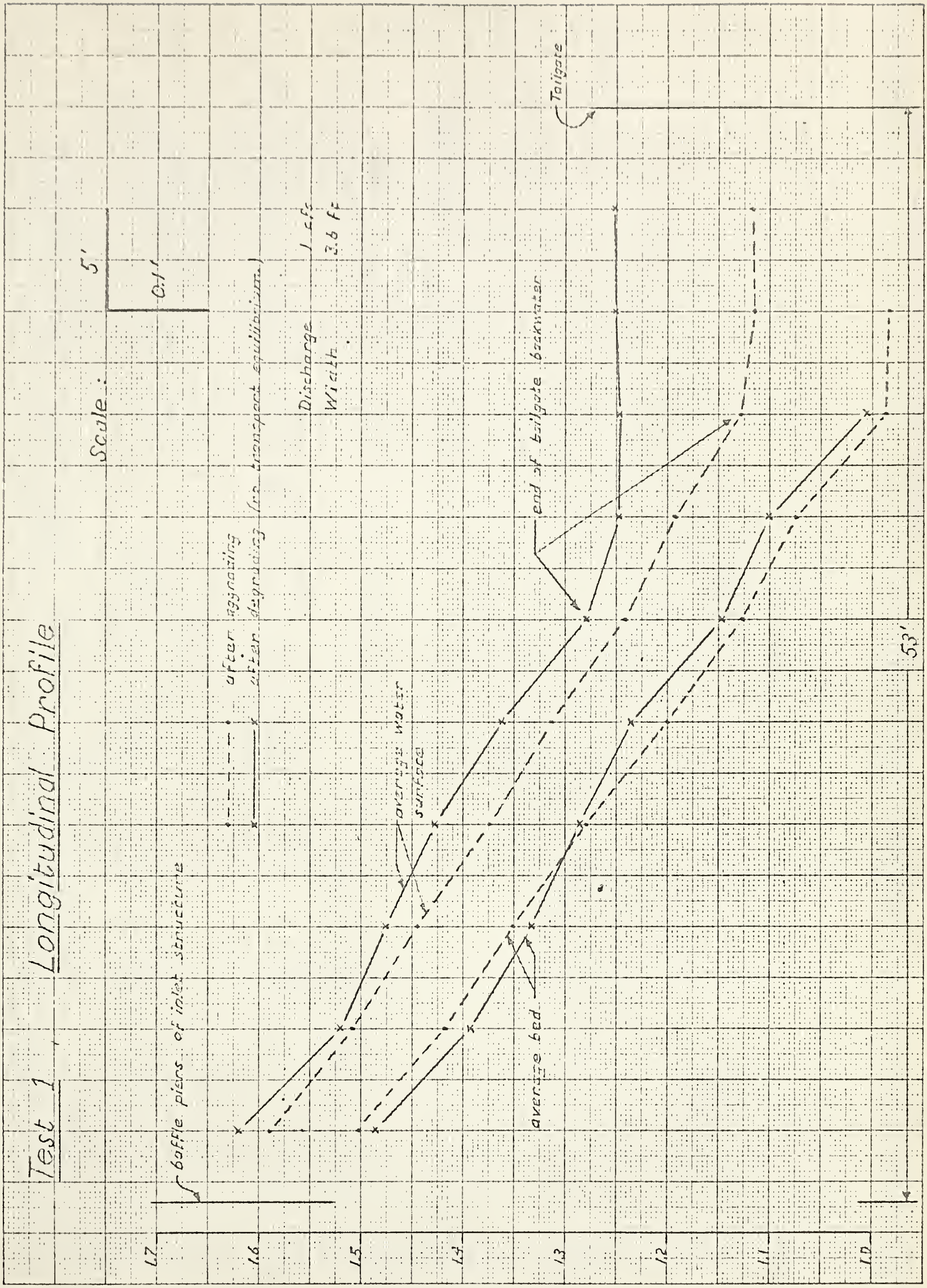

Test 1 Longitudinal Profile
Scale:
5'
0.1'
Discharge 1 cfs
Width 3.6 ft
after aggrading
after degrading (to transport equilibrium)
baffle piers of inlet structure
average water surface
average bed
end of tailgate breakwater
Tailgate
53'
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0

Fig. 54

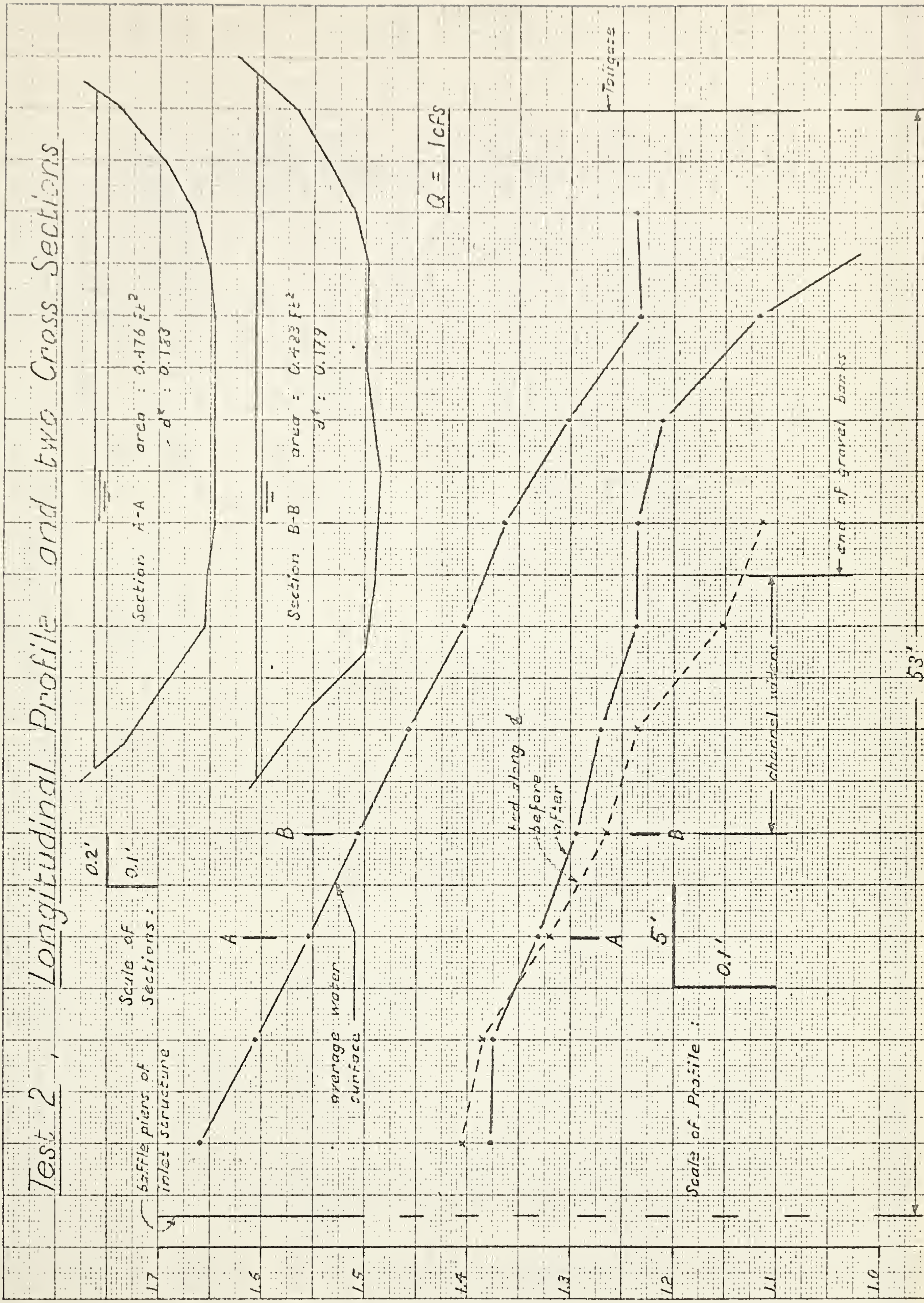

Test 2, Longitudinal Profile and two Cross Sections
Q = 1cfs
Section A-A
area : 0.476 ft²
d : 0.183
Section B-B
area : 0.423 ft²
d : 0.179
Scale of Sections :
0.2'
0.1'
baffle piers of inlet structure
average water surface
bed along ℄
before
after
A
B
A
B
Scale of Profile :
5'
0.1'
channel widens
end of gravel banks
Tailgate
53'
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0

Fig. 55

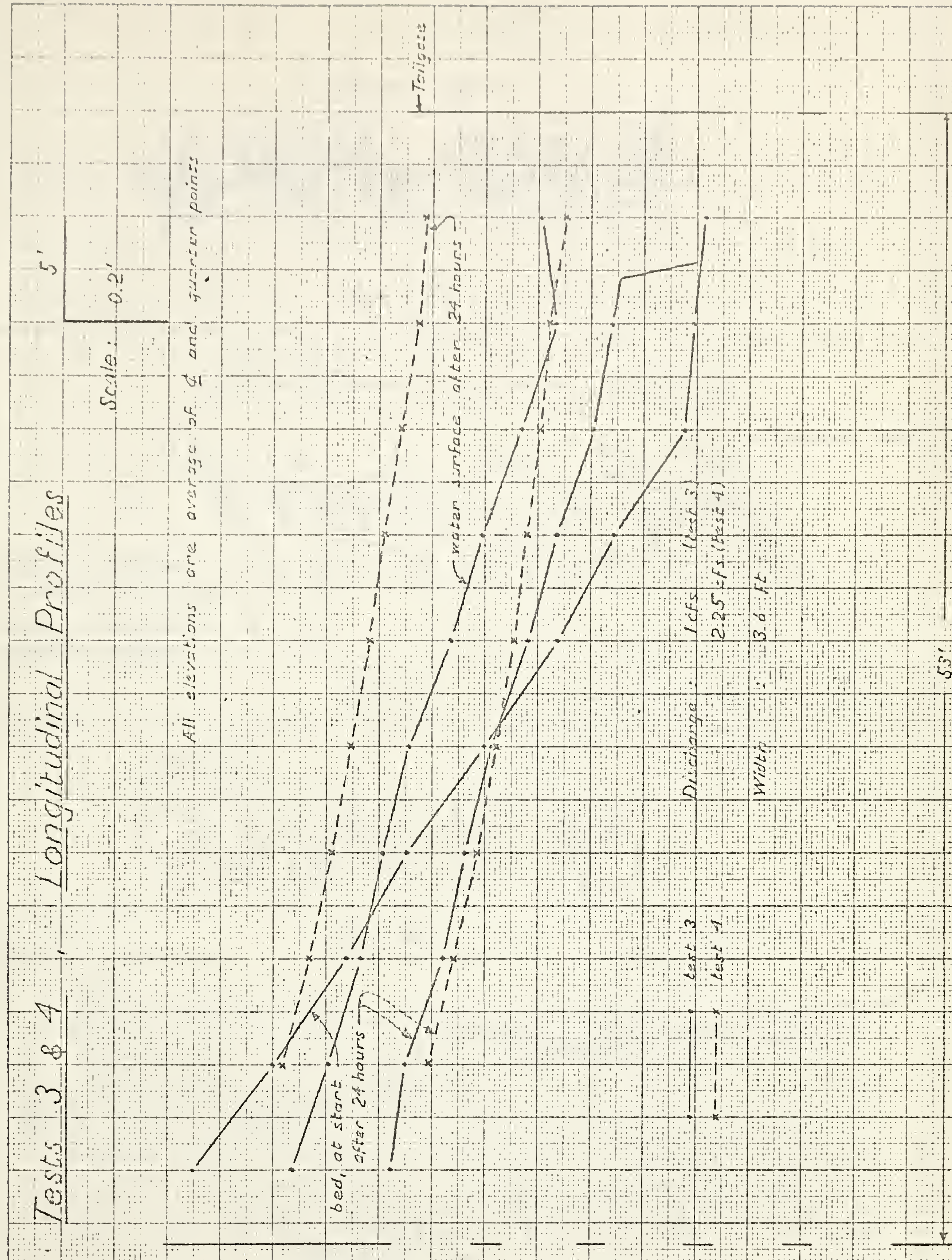

Tests 3 & 4, Longitudinal Profiles
Scale: 0.2'
5'
All elevations are average of ℄ and quarter points
water surface after 24 hours
bed, at start
after 24 hours
Tailgate
Discharge : 1 cfs (test 3)
2.25 cfs (test 4)
Width : 3.6 ft
test 3
test 4
55'
1.8
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7

Fig. 56

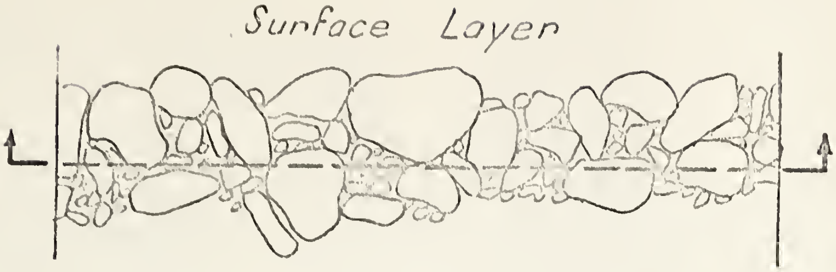


Fig. 57

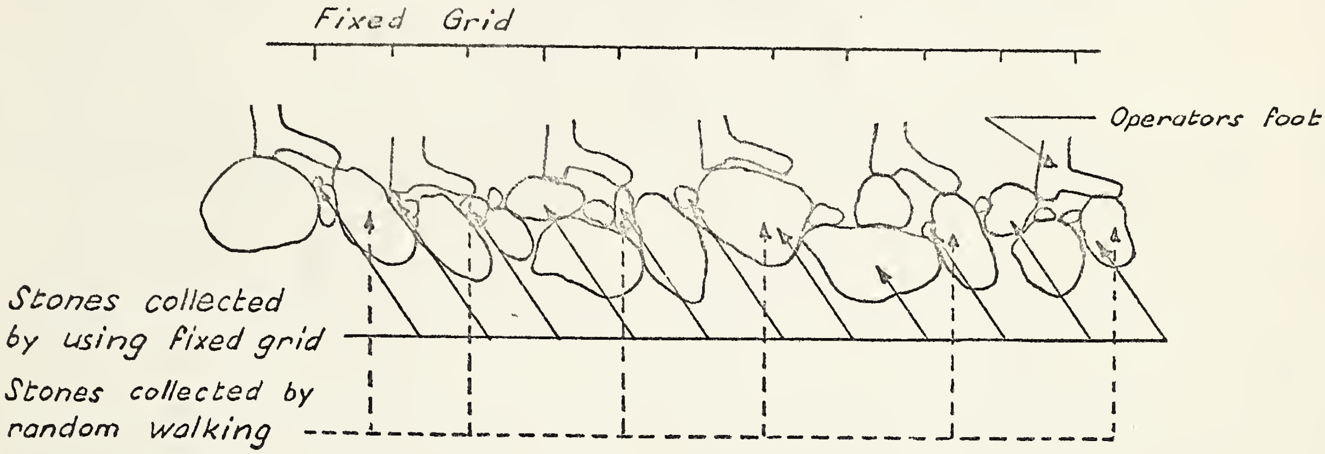


Fig. 58

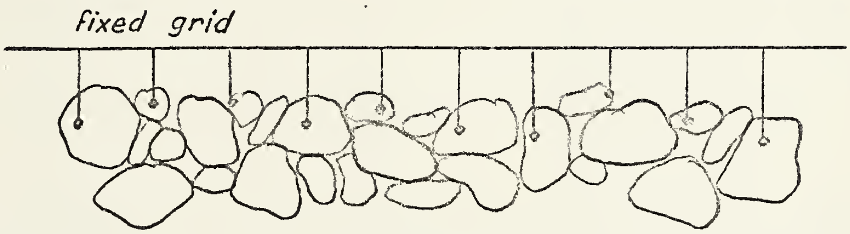


Fig. 59

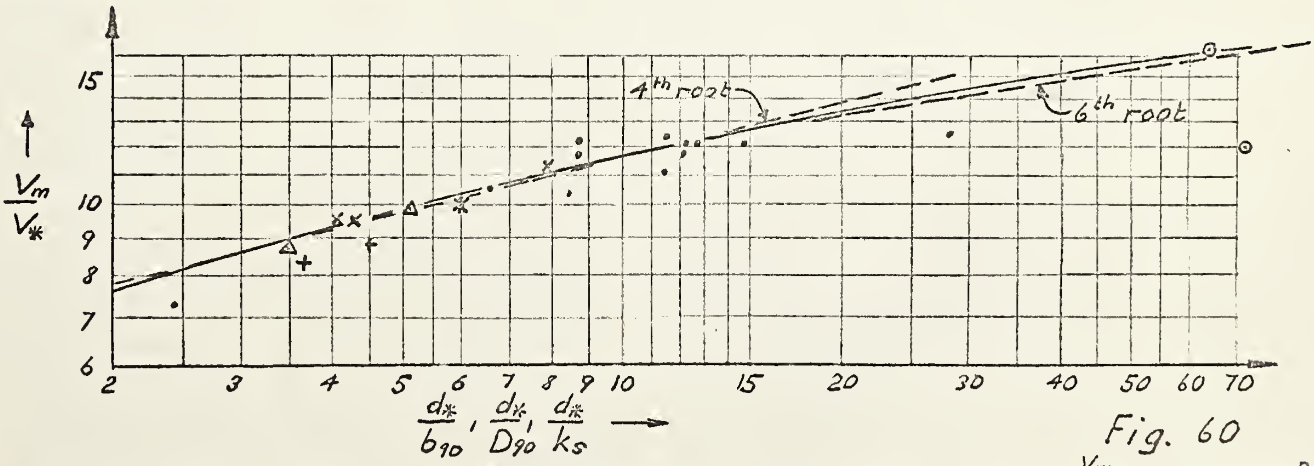
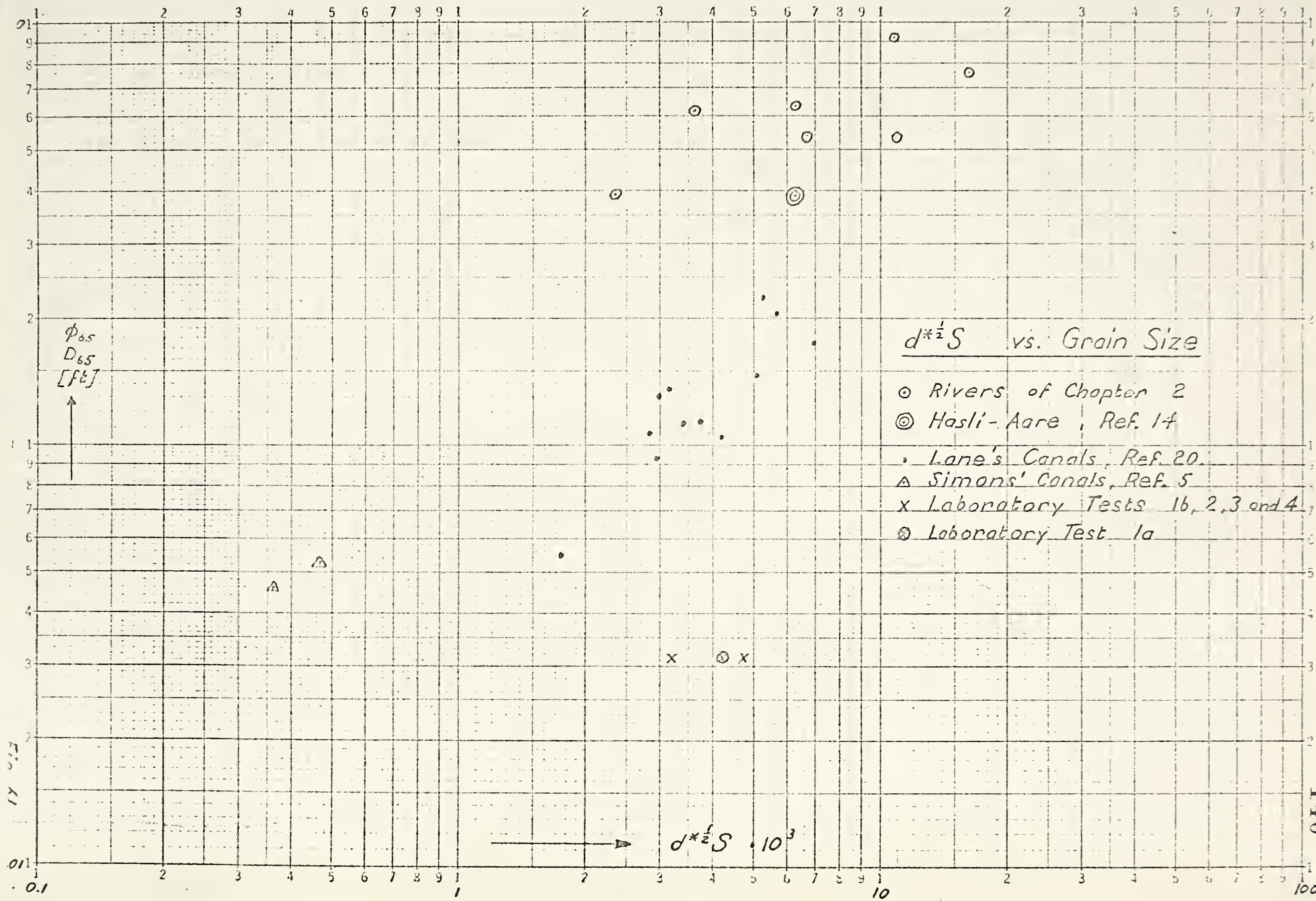
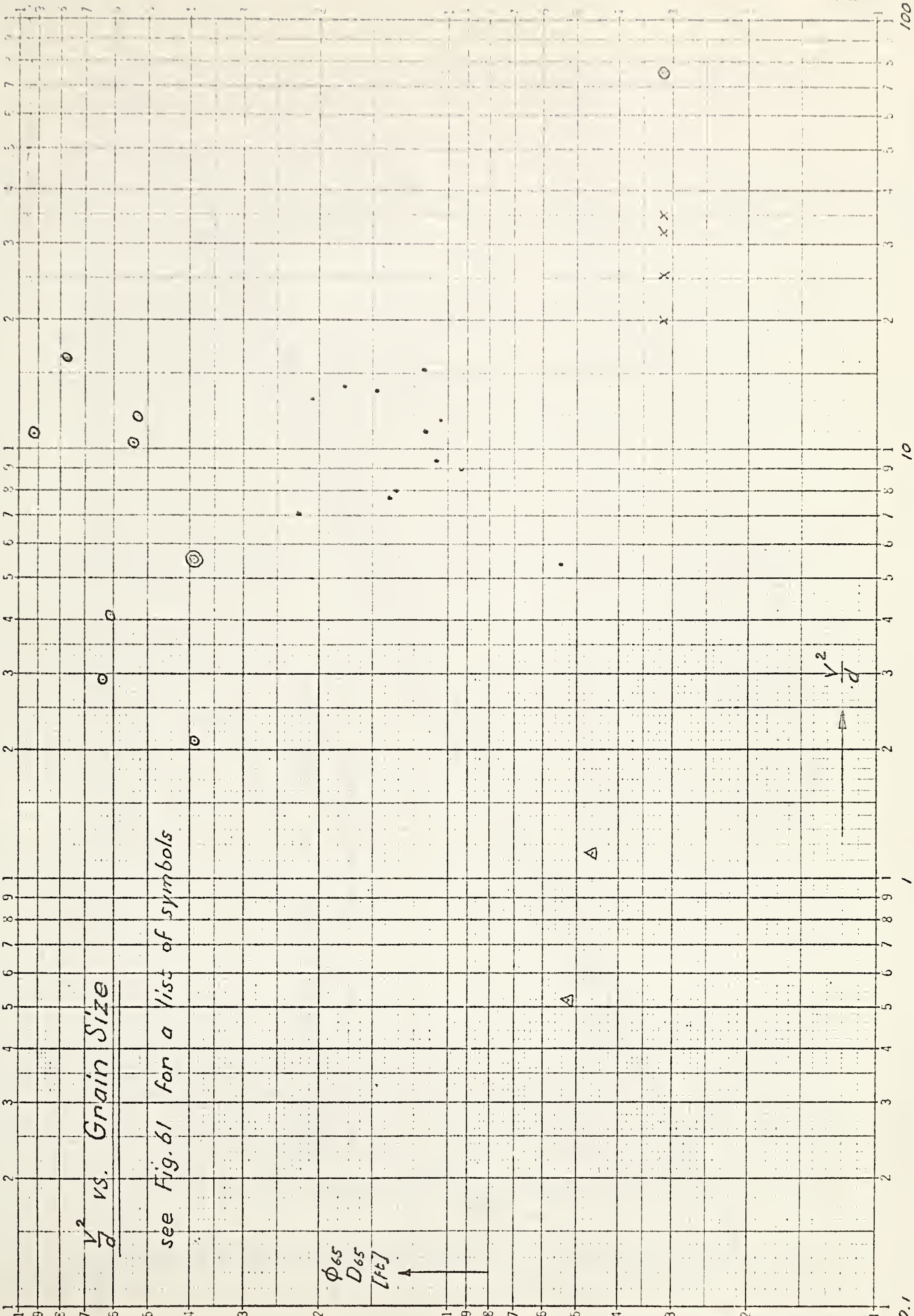


Fig. 60

• San Luis Valley Canals
⊙ Simons - Bender Canals
× Laboratory Tests 1, 3, 4.
△ Quesnel at Lawless Creek
+ Chilko at Henry's Crossing
∗ Hasli - Aare

—— $\frac{V_m}{V_*} = 5.66 \log 11.2 \frac{R}{k_s}$
– – – exponential equ.'s


$d^{*\frac{1}{2}}S$ vs. Grain Size
Rivers of Chapter 2
Hasli-Aare, Ref. 14
Lane's Canals, Ref. 20.
Simons' Canals, Ref. 5
Laboratory Tests 1b, 2, 3 and 4
Laboratory Test 1a
ϕ_{65} D_{65} [ft]
$d^{*\frac{1}{2}}S \cdot 10^3$
0.1
1
10
100
.01
.1
1

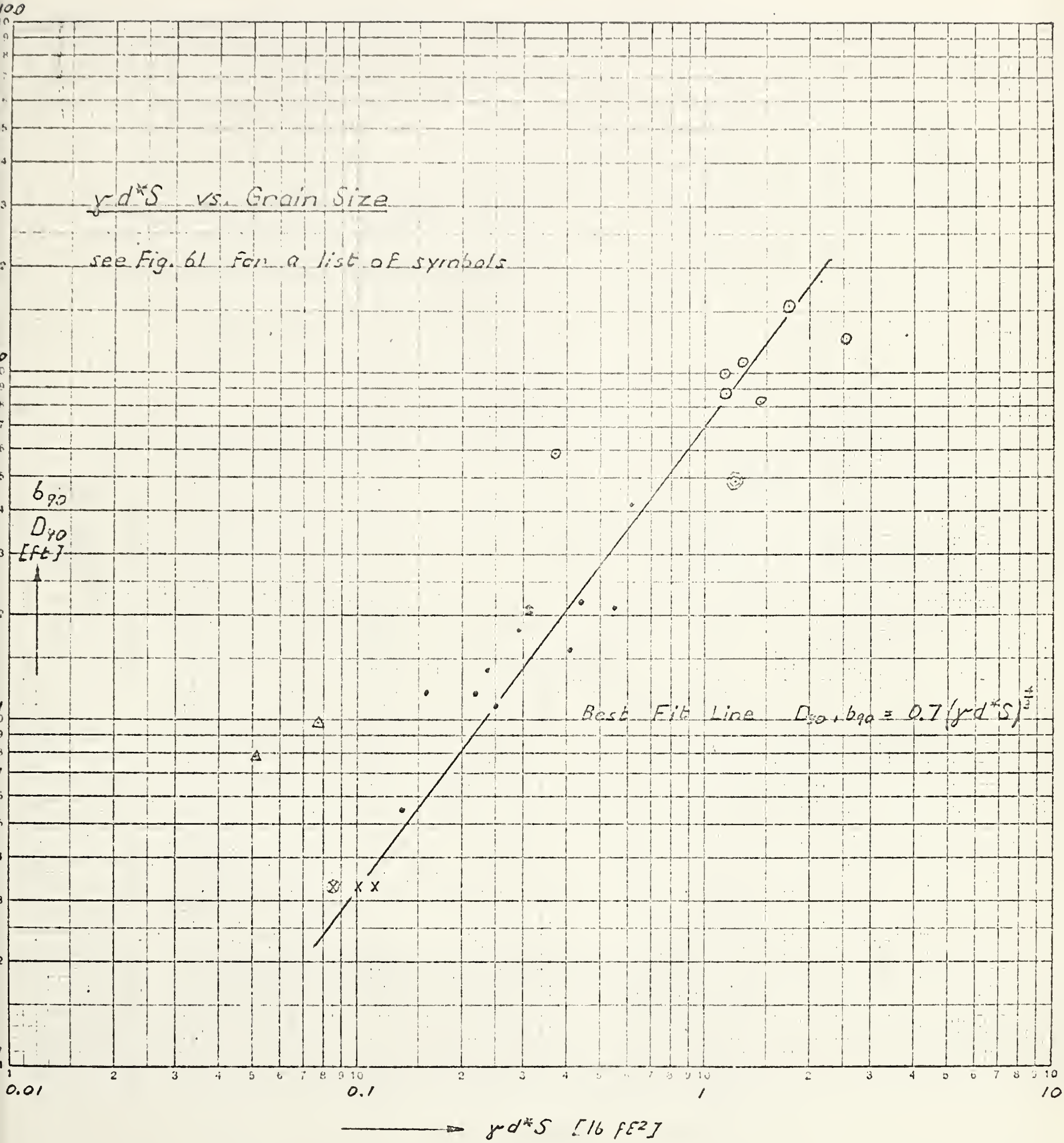


V²/d vs. Grain Size
see Fig. 61 for a list of symbols
Φ65
D65
[ft]
V²/d
1.0
0.1
0.01
0.1
1
10
100

Fig. 62

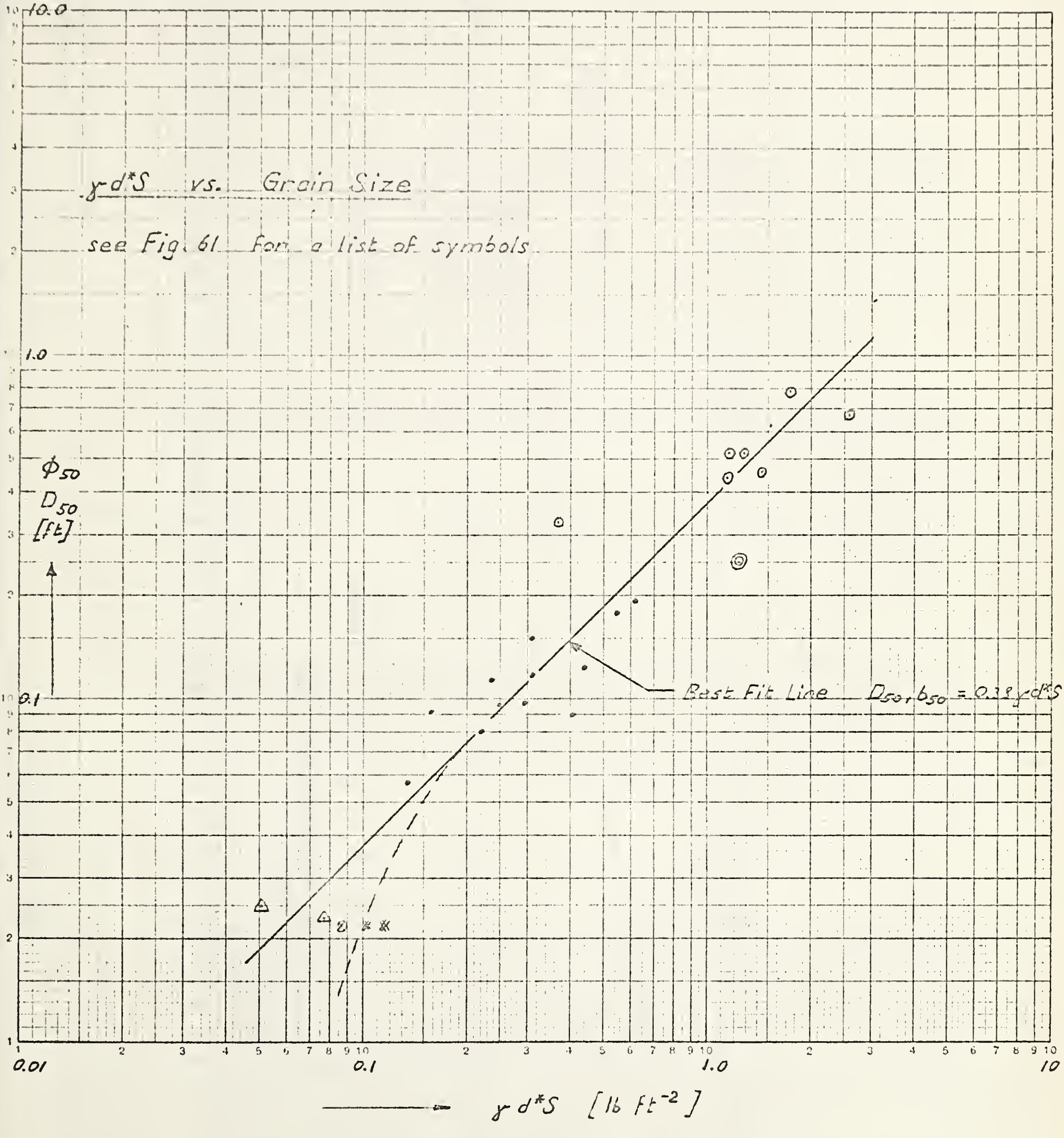
Water Surface Width vs. Discharge
see Fig. 61 for a list of symbols
Lacey's eqn. 5.2.6
Ws = 1.95 Q^½
Best Fit Line
Ws (ft)
Q in cfs
1.0
10
100
1,000
10,000
100,000

Fig. 63



γd*S vs. Grain Size
see Fig. 61 for a list of symbols
Best Fit Line $D_{90}, b_{90} = 0.7 (\gamma d^* S)^{\frac{4}{3}}$
b_{90}
D_{90}
[ft]
0.01
0.1
1
10
γd*S [lb ft²]

Fig. 64



γd^*S vs. Grain Size
see Fig. 61 For a list of symbols
Φ_{50}
D_{50}
[ft]
Best Fit Line $D_{50}, b_{50} = 0.33\,\gamma d^*S$
10.0
1.0
0.1
0.01
0.1
1.0
10
γd^*S [lb ft^{-2}]

Fig. 65

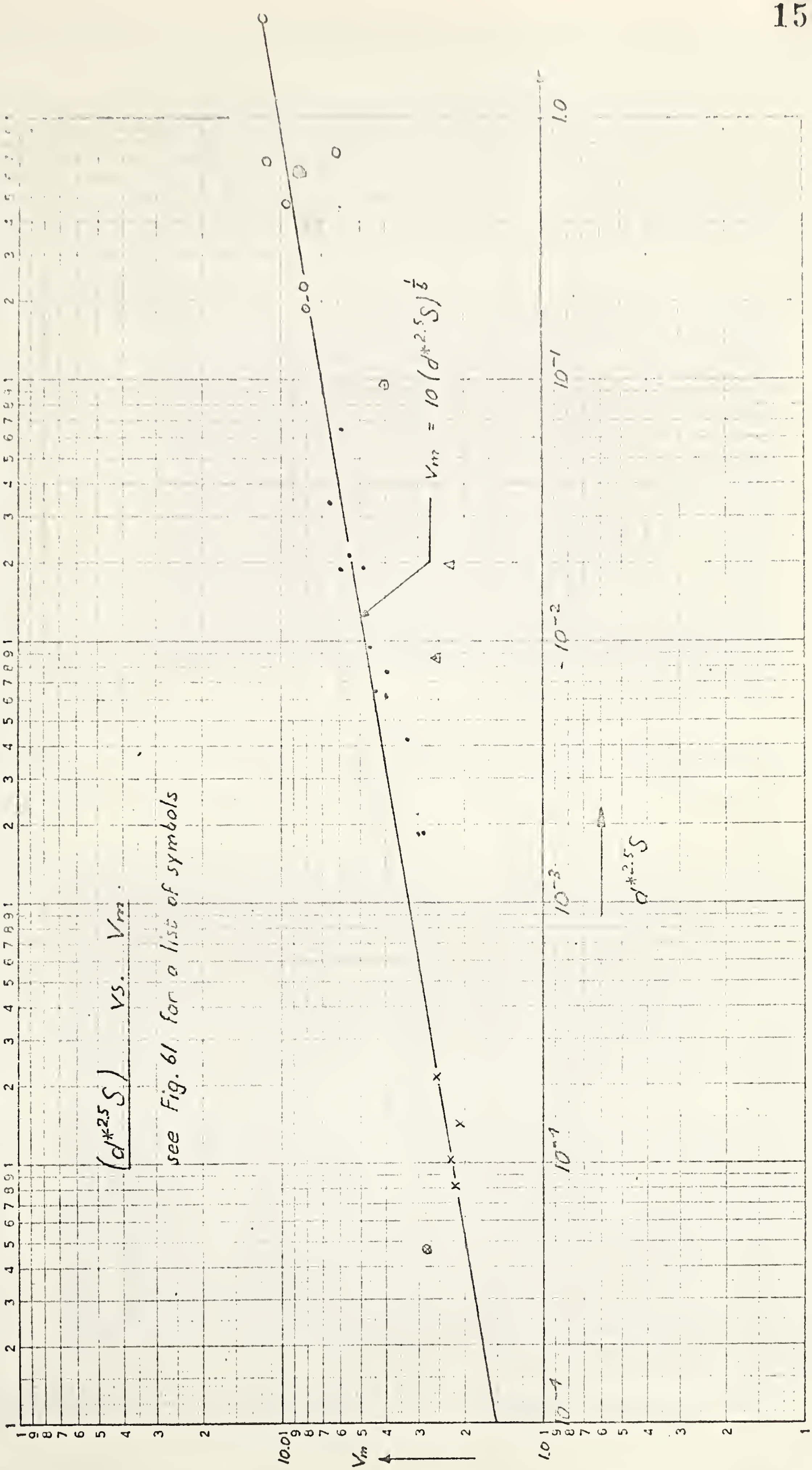


Fig. 66

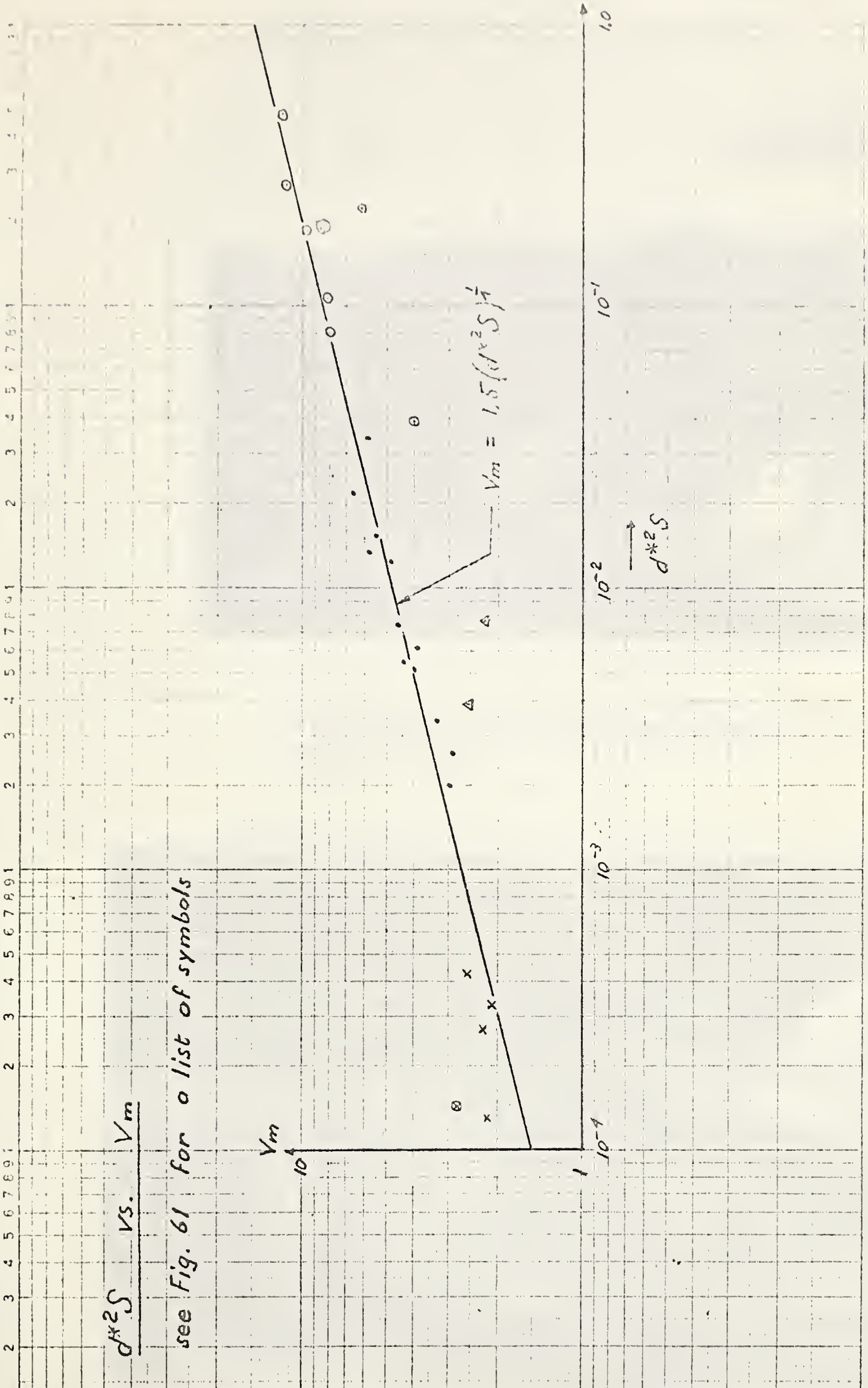

d*2S vs. Vm
see Fig. 61 for a list of symbols
Vm = 1,5 (d*2S)^1/4
Vm
10
1
10^-4
10^-3
10^-2
10^-1
1.0
d*2S

Fig. 67

QUESNEL RIVER AT LAWLESS CREEK

Picture 1 Looking down from road between Quesnel Forks and Likely on to the upper part of the measuring reach.
July 3, 1962.

Picture 2 As picture 1 but on Sept. 20, 1962, at low river stage.

Picture 3 Area of gravel sample 4, July 4, 1962.

CARIBOO RIVER AT QUESNEL FORKS

Picture 4 Looking over the measuring reach from st.4, l.b., on June 30, 1962.

Picture 5 Looking upstream from st.3 on Sept. 18, 1962.

Picture 6 Tape stretched out for gravel sample 1, Sept. 19, 1962.

Picture 7 Road cut near st.4. The material for sieve curve 1 was taken here.

CARIBOO RIVER AT OUTLET OF CARIBOO LAKE

Picture 8 Looking downstream from st.5 on July 10, 1962.

Picture 9 Typical bank at high stage.

Picture 10 Place of gravel sample 5 near st.3, on bar in centre of river.

Picture 11 Gravel sample 3, at waterline, st.3, r.b.

TASEKO RIVER BELOW TASEKO LAKE

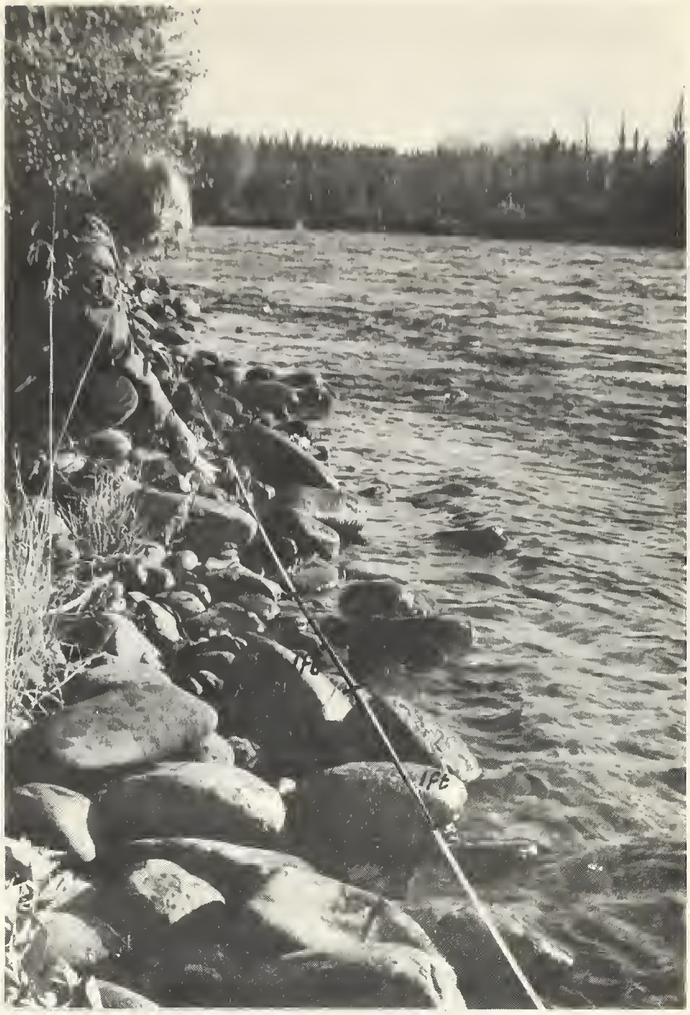
Picture 12 Looking downstream along the measuring reach from a place between st.1 and st. 2, r.b. Gravel sample 5 was taken here. Aug. 19, 1962.

Picture 13. Looking downstream from st. 4, l.b. Gravel sample 3 was taken on bar in foreground. Aug. 21, 1962.

Picture 14 Obstructed part of the river at the upper end of the measuring reach. Tape in place for gravel sample 2. Aug. 19, 1962.

Picture 15 Lingfield Creek in September.

Picture 16 Looking upstream from near st. 5, l.b.
Gravel sample 5 was taken from the almost
exposed bar in the river. Sept. 14, 1962.



Picture 17 Location of gravel sample 6 at st. 4, r.b. Sept. 12, 1962.

Picture 18 Material exposed in roadcut through river bank. (Sieve curve 2, gravel sample 7)

CHILKO RIVER AT THE OUTLET OF CHILKO LAKE

Picture 19   Looking downstream from st. 2, r.b. on July 23, 1962.

THOMPSON RIVER AT THE OUTLET OF KAMLOOPS LAKE

Picture 20   Looking upstream at the gravel bar on which the gravel sample of Fig. 43 and of picture 21 were taken.

Picture 21   Bed pavement on bar near st. 10, r.b.

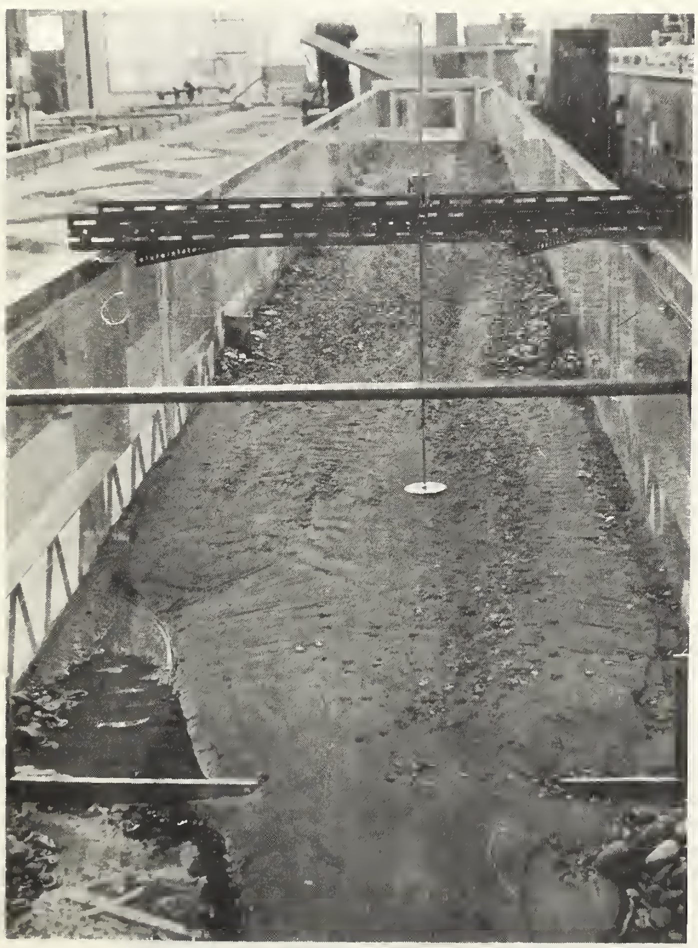
LABORATORY WORK

Picture 22   The flume before the first test.

Picture 23 The upper part of the flume during test 1, aggrading.

Picture 24 The dewatered flume after 2 h of aggrading (test 1)

Picture 25 The bed near the upstream end of the flume after aggrading. (test 1)



Picture 26 Dewatered flume after aggrading.(test 1). Point gauge set for measuring bed elevation.

Picture 27 Close up of bed pavement. Flow was left to right.

Picture 28 Artificial channel of test 2.

Picture 29 The channel formed during test 2.